ARIZONA DEPARTMENT OF TRANSPORTATION

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HILFIKER REINFORCED SOIL EMBANKMENT WITH FULL-HEIGHT, CAST-IN-PLACE CONCRETE PANELS

Final Report

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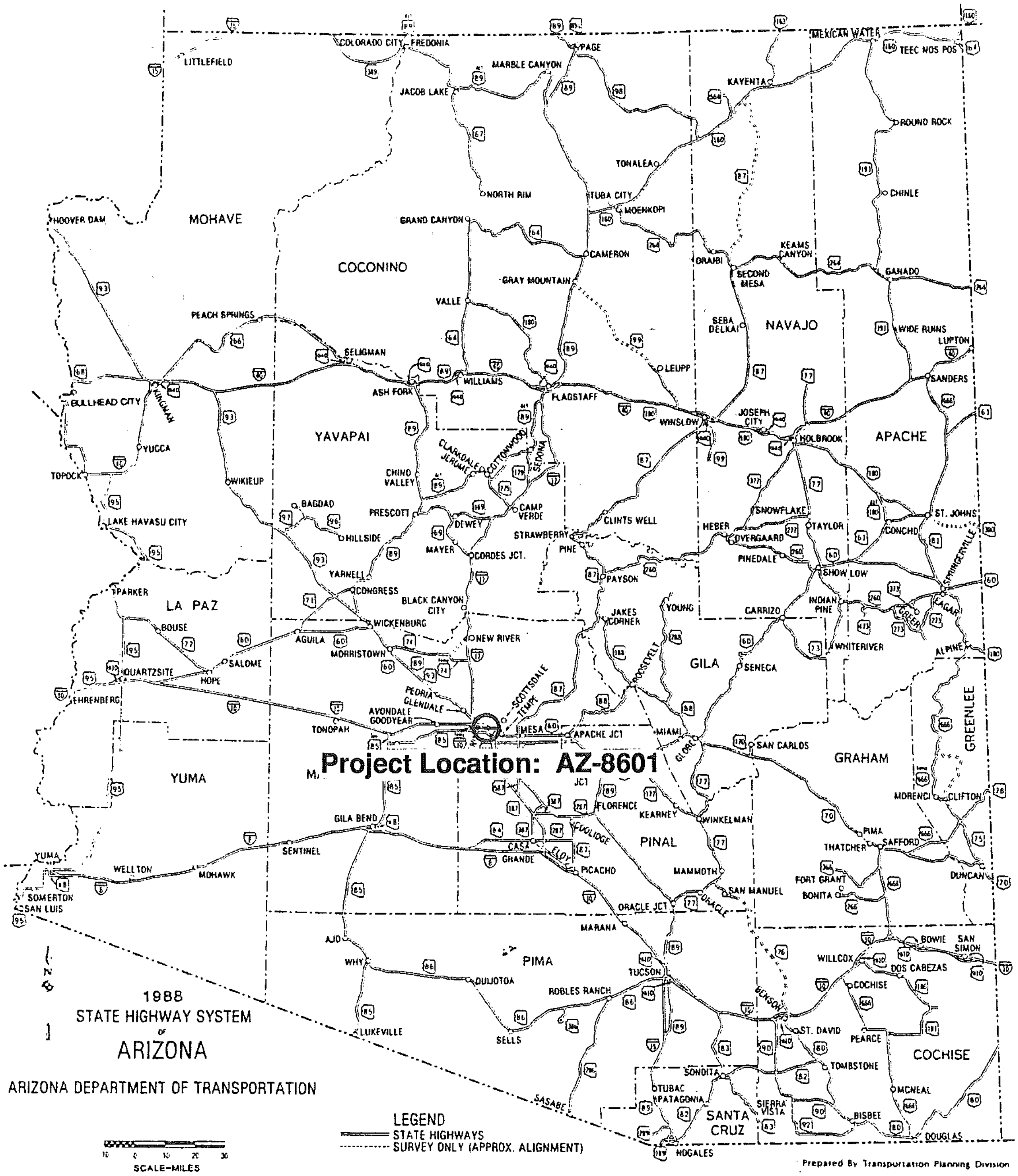

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16. Abstract

The objective of this experimental project was to evaluate the construction and performance of a full-height retaining wall system. The contractor chose to use the Hilfiker Reinforced Soil Embankment with cast-in-place, concrete panels. The project included three Hilfiker walls at the interchange of I-10 and 24th St. in Phoenix.

Construction of the embankment consisted of placing steel mesh reinforcing mats at 2' intervals, followed by placing and compacting the backfill material. Some of the backfill had to be imported as the on-site material failed resistivity tests. The concrete panels were poured after the embankment was completed. The concrete was poured from the top of the embankment, many times in excess of 20'. Because of the pour height, and difficulty in vibrating, air voids, hairline cracks, and honeycombing were prevalent in the northeast and southwest walls.

The evaluation period began with the construction of the walls, September 1986, and extended through October, 1991. The southeast wall is significantly distressed. A large vertical crack has formed along the Hilfiker wall - bridge abutment interface. Also, the anchor slab above the crack has been witnessed deflecting considerably (about 1"). A hypothesis tied to these problems is that a void may have formed beneath the anchor slab. Water exiting the embankment may be carrying soil out with it. An erosion channel immediately below the crack is evidence of water passing through the crack. The other walls also share these distresses to a much lesser degree. ADOT is currently investigating the cracks and deflections.

Recommendations for future use of reinforced earth systems include increasing backfill density requirements and providing controlled drainage from the embankment. The drainage should be filtered to prevent the removal of backfill material. Also, during design, provision should be made for the geometric configurations of the features of the roadway, such as catch basins and light pole foundations.

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INTRODUCTION

Background

Many soils that are adequately compacted can have high compressive and shear strengths. However, soils rarely have any significant tensile strength. Mechanically stabilized embankments include reinforcement that interacts with the soil by means of friction, and thereby increases the tensile strength and overall stability of the soil. Reinforcement materials used include steel rods and bars, as well as geotextiles and fibers.

The bond between the reinforcement and the soil is a shear bond that is dependent on the vertical effective stress in the soil at any given depth. Because effective stress varies with pore water pressure, the use of reinforcement in poorly draining silts or clays leads to unpredictable performance. Hence, granular soils such as sand are commonly used for mechanically stabilized embankments. Granular soils are essentially free draining, which leads to rapid dissipation of any pore water pressure.

The overall stability of soil used for structural purposes is judged considering the possibility of sliding, overturning, bearing capacity failure, and excessive settlement. Considerations regarding the stability of mechanically stabilized soil also include safety against reinforcement pullout and rupture, and reduction of effective reinforcement area due to corrosion.

Problem Statement

During the last decade, there has been a rapid increase in the number of mechanically stabilized retaining systems commercially available. This has left government agencies with the dilemma of weighing the potential for significant initial savings versus a frequently unknown product performance record.

There has been considerable experience gathered regarding retaining wall systems that use small modular precast facing panels of approximately 25 sq. ft. Such systems have the panels individually attached to the reinforcing elements in the ground. These panels have been shown to allow considerable soil settlement without significant distress. Recently, however, aesthetic considerations have led to a preference for full-height retaining wall panels. The Arizona Department of Transportation (ADOT) had specified full-height retaining walls with an architectural rustication detail for Interstate 10 in the metropolitan Phoenix area. This was done to maintain uniformity in the urban highway system.

There are concerns with the use of full-height panel retaining walls. Full-height panels require many connections between the individual panels and reinforcing elements, which in turn leads to indeterminacy in the structure. There is not a well-known effective means to analyze the internal behavior of full-height panel, mechanically stabilized systems. Also, there is a paucity of performance data available.

OBJECTIVE

The objective of this experimental project was to monitor the construction and performance of a Hilfiker full-height retaining wall system. The construction of the system was documented with notes on progress, methods of construction, and difficulties encountered. The performance of the wall, based on surveyed movement and visual inspection, was recorded and evaluated.

Due to the limited experience and knowledge of full-height panel retaining walls, the Federal Highway Administration (FHWA) requested that the three permanent retaining systems to be installed at the interchange of I-10 and 24th Street in Phoenix, Arizona, be classified as an experimental project. The Arizona Department of Transportation approved three different wall systems for use at this location. Those systems were:

1. Reinforced Earth System with precast concrete face panels and cast-in-place coping.

2. Retained Earth System with precast concrete face panels and cast-in-place coping.

3. Hilfiker Reinforced Soil Embankment with cast-in-place concrete facing.

The contractor chose to use the Hilfiker system because it was believed to be the least expensive to construct. The FHWA approved workplan for this experimental project is in accordance with the FHWA Geotechnical Advisory 5.0.3 and is included in this report as APPENDIX A.

PROJECT LOCATION AND DESCRIPTION

ADOT highway project IR-10-3(204) included the construction of the Hilfiker full-height retaining walls. The project began at 16th Street and extended east to 28th Street (I-17 MP 195.09 to I-10 MP 150.39), a distance of 1.73 miles. Figure 1 shows the locations of the Hilfiker walls.

The project consisted of construction of a traffic interchange between I-10, I-17, and 24th Street. Included in this project was the construction of three permanent retaining walls. The walls are identified by the relative position; the northeast (NE) wall, the southwest (SW) wall, and the southeast (SE) wall. These walls became the Experimental Project incorporating the Hilfiker Reinforced Soil Embankment System, designated AZ-8601.

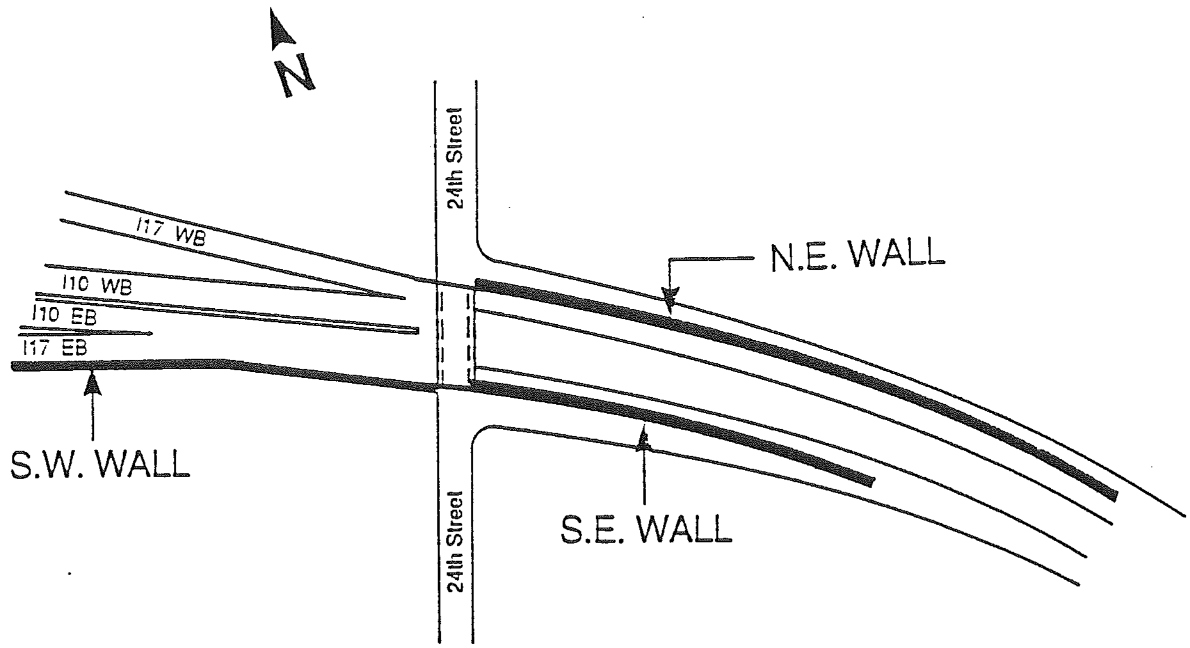


Figure 1 Location of Hilfiker Walls

Climatic Conditions

Precipitation in Phoenix averages 7.01 inches per year with the monthly distribution depicted in Figure 2[1]. Figure 3 shows the average high and low daily temperature variations by month.

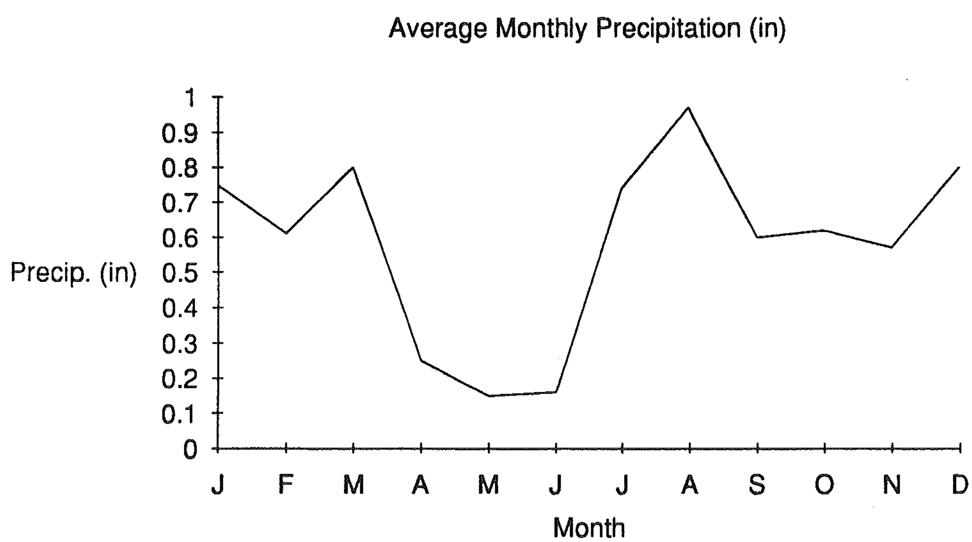


Figure 2 Average Monthly Precipitation for Phoenix, Arizona[2].

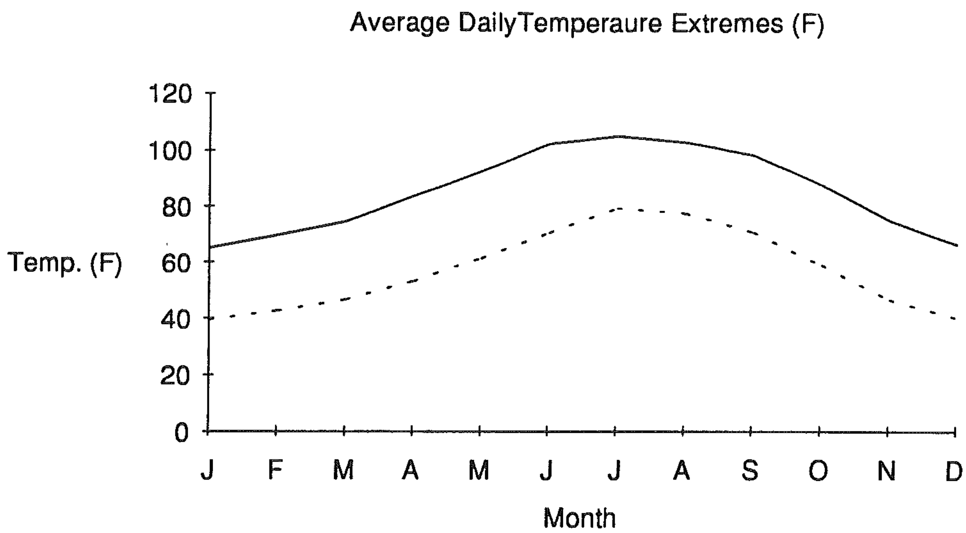


Figure 3 Average Daily Temperature Extremes by Month for Phoenix, Arizona[3].

Soil Conditions

There was a geotechnical investigation consisting of four test pits and two drilled boreholes conducted prior to construction of the southwest wall. The pits and boreholes were placed along the site of the retaining walls. The excavation logs of these tests are presented in APPENDIX B and are summarized in Table 1. Excavation data for the northeast and southeast wall sites was not available.

CONSTRUCTION

Tanner Construction was awarded the contract for the construction of the Hilfiker Reinforced Soil Embankments of this project. Selvage, Heber, and Nelson were the consulting engineers for the retaining wall design.

The construction of the embankments and the panels for each of the walls was similar, and a description is forthcoming. A construction guide provided by Hilfiker is included as APPENDIX C, and reductions of a portion of the construction project plans are included as APPENDIX D. Further comments or conditions specific to a wall are addressed individually under that wall's heading.

Embankment Construction

Preparation of the subgrade consisted of excavating the existing soil to design level. The soil was then tested for resistivity. If the soil did not pass resistivity requirements, the excavation was continued one more foot, and an approved aggregate base course (ABC) was placed and compacted to bring the elevation up to plan level. ADOT specifications for backfill material are given in APPENDIX E. The final grade of the subgrade varied between one and two percent, sloping away from the face of the wall.

Identification	Depth	Description
Test Pit #1	0 to 2 ft	Gravelly sand and fill.
	2 to 11 ft.	Sand, gravel, cobbles, and some boulders.
Test Pit #2	0 to 2 ft.	Gravelly sand and fill.
	2 to 18 ft.	Sand, gravel, cobbles, with large concrete blocks, asphalt, wood and metal scrap.
Test Pit #3	0 to 2 ft.	Gravelly sand and fill.
	2 to 14 ft.	Sand, gravel, cobbles, boulders, concrete blocks, and wood and metal scrap.
	14 to 19 ft.	Sand, gravel, cobbles, and some boulders.
Test Pit #4	0 to 2 ft.	Gravelly sand and fill.
	2 to 8 ft.	Sand, gravel, cobbles, boulders, concrete blocks, and wood and metal scrap.
	8 to 13 ft.	Sand, gravel, cobbles, and some boulders.
Boring #84-135	0 to 1 ft.	Silty sand with clay.
	1 to 3 ft.	Gravel and cobbles
Boring #84-136	0 to 2 ft.	Gravelly sand and fill
	2 to 8 ft.	Sand, gravel, cobbles, and boulders mixed with debris.
	8 to 13 ft.	Sand, gravel, cobbles, and some boulders.

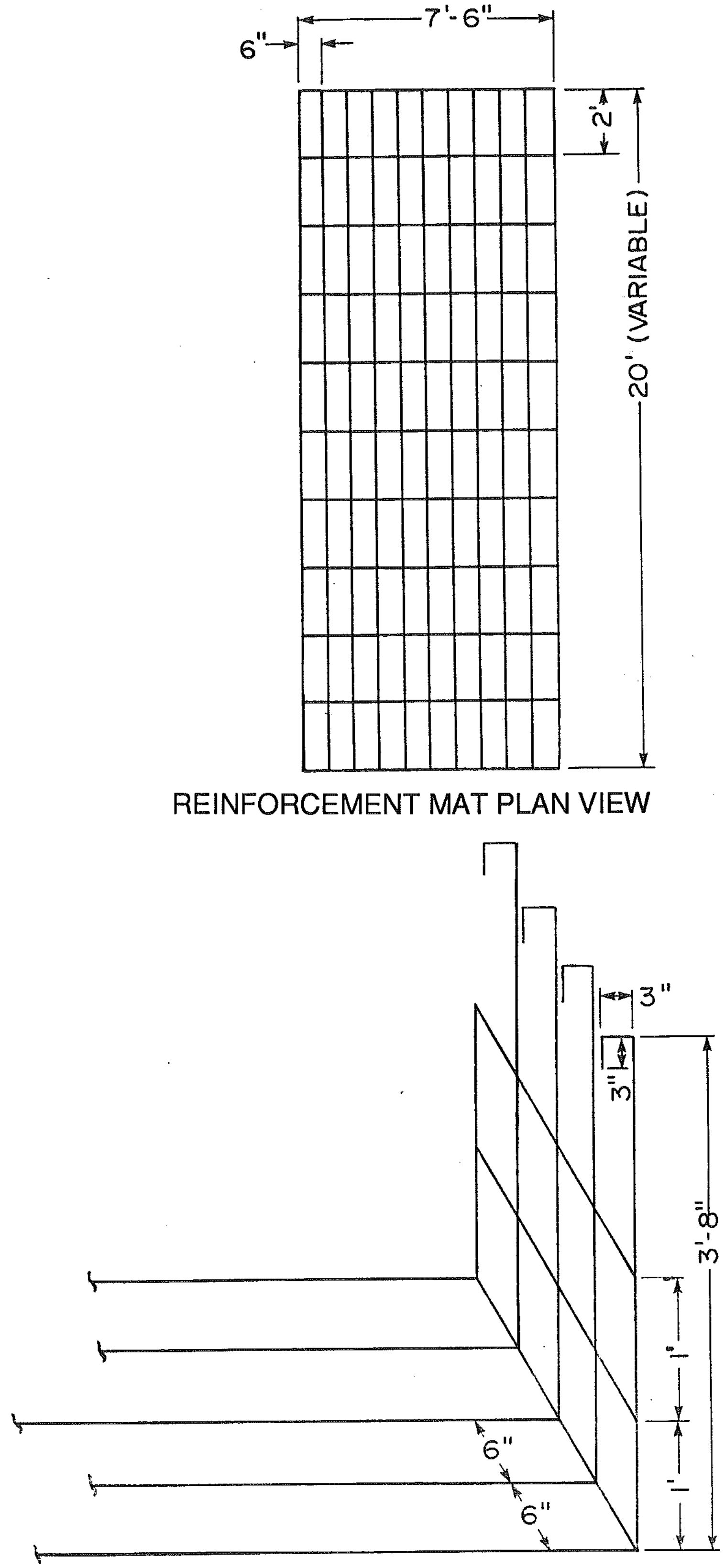
Table 1 Summary of Geotechnical Logs for Southwest Wall Site.

A reinforced concrete leveling course was constructed to serve as the base for the cast-in-place concrete faces of the Hilfiker walls. The concrete course is 15" wide and 6" deep. Figure 4 is a photograph of the preparation of the leveling course and subgrade.

Figure 4 Preparation of Subgrade and Leveling Course.

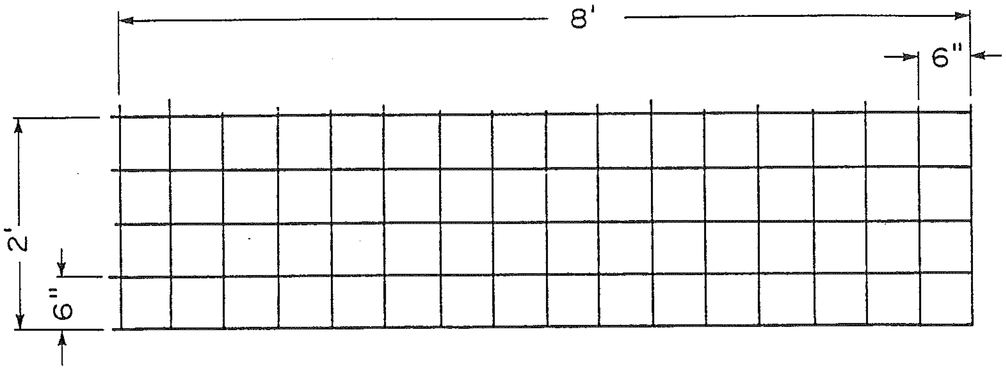
Reinforcement of the embankment was in the form of galvanized welded steel wire mesh (W7xW7) mats. The horizontal mats were 7.5' x 20' (variable), and the backing mats were 8' x 2'. Figures 5 and 6 show dimensions of the reinforcing mats. Figure 7 demonstrates the placement of the first layer of reinforcement. The first layer was placed on the subgrade and held in place with form pins. A spacer cage was used on the first layer to hold the backing mat away from the vertical bars of the horizontal reinforcing mat.

A geotextile fabric screen was used to keep the compacted backfill behind the wire mesh. Initially, the fabric was rolled under the reinforcing mats, resulting in a 1/2" to 3/4" crevice. Because of the concern that concrete may not have filled the crevice, the fabric was cut into 1/2' intervals and placed on the vertical bars of the mats. This is shown in Figure 8.

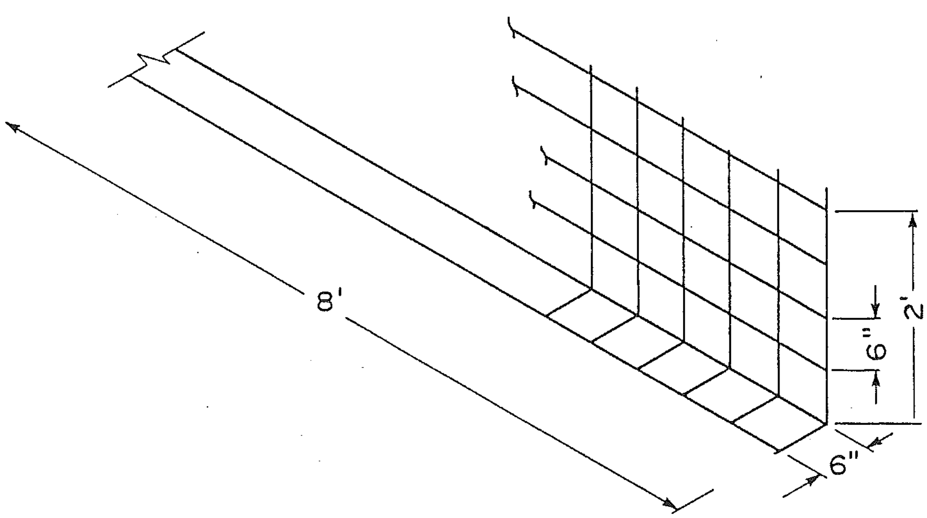


REINFORCEMENT MAT PERSPECTIVE VIEW

Figure 5 Typical Dimensions of the Horizontal Reinforcing Mats.



BACKING MAT PLAN VIEW



BACKING MAT PERSPECTIVE VIEW

Figure 6 Typical Dimensions of the Backing Mats.

Figure 7 Placement of the First Layer of Reinforcement.

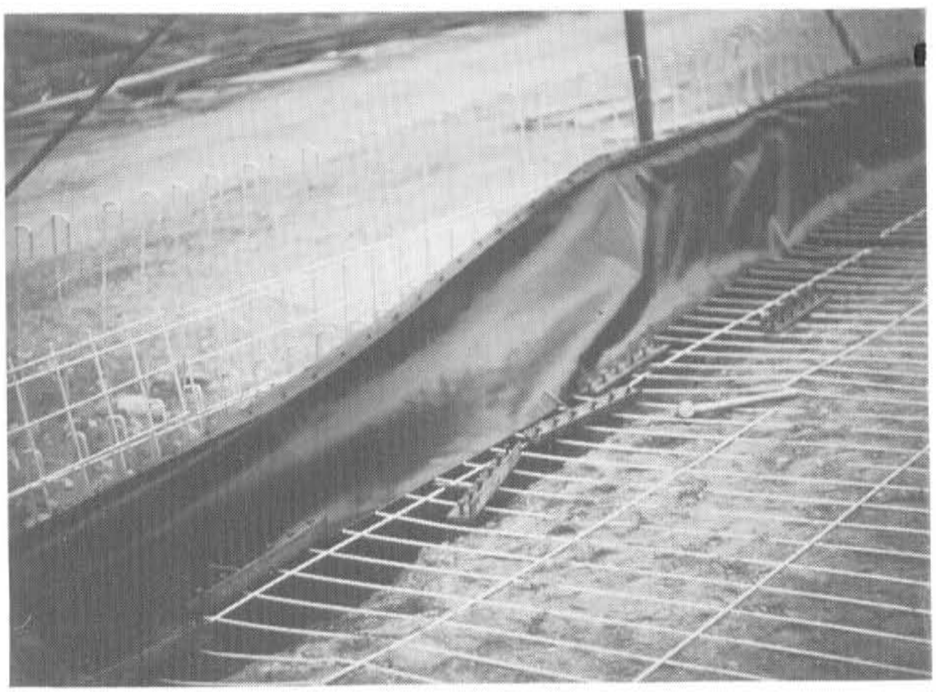


Figure 8 Geotextile Fabric Screen to Retain Compacted Backfill.

The horizontal reinforcing mats were placed successively in the embankment at 2' lifts. Each lift included a non-galvanized #4 rebar to mitigate thermal expansion or contraction of the wire mesh. The backfill material was placed in 1' lifts on the reinforcing mats with a ten-wheel dump truck and compacted with passes by a rock-bucket loader, a grader, a backhoe, water trucks, and dump trucks. Compaction of the backfill within 2' of the backing mats was performed by water-jetting and a hand vibrating tamper (a jumping jack). Initially, hand rammers were used in this zone, but they were found to be slow and inefficient. A schematic diagram of the two lower 1' lifts of the Hilfiker embankment is shown in Figure 9. Figure 10 shows the use of a hand tamper near the backing mat, and a dump truck used to deliver the backfill material. Figure 11 is the backfill material being distributed on the reinforcing mats with a backhoe/loader.

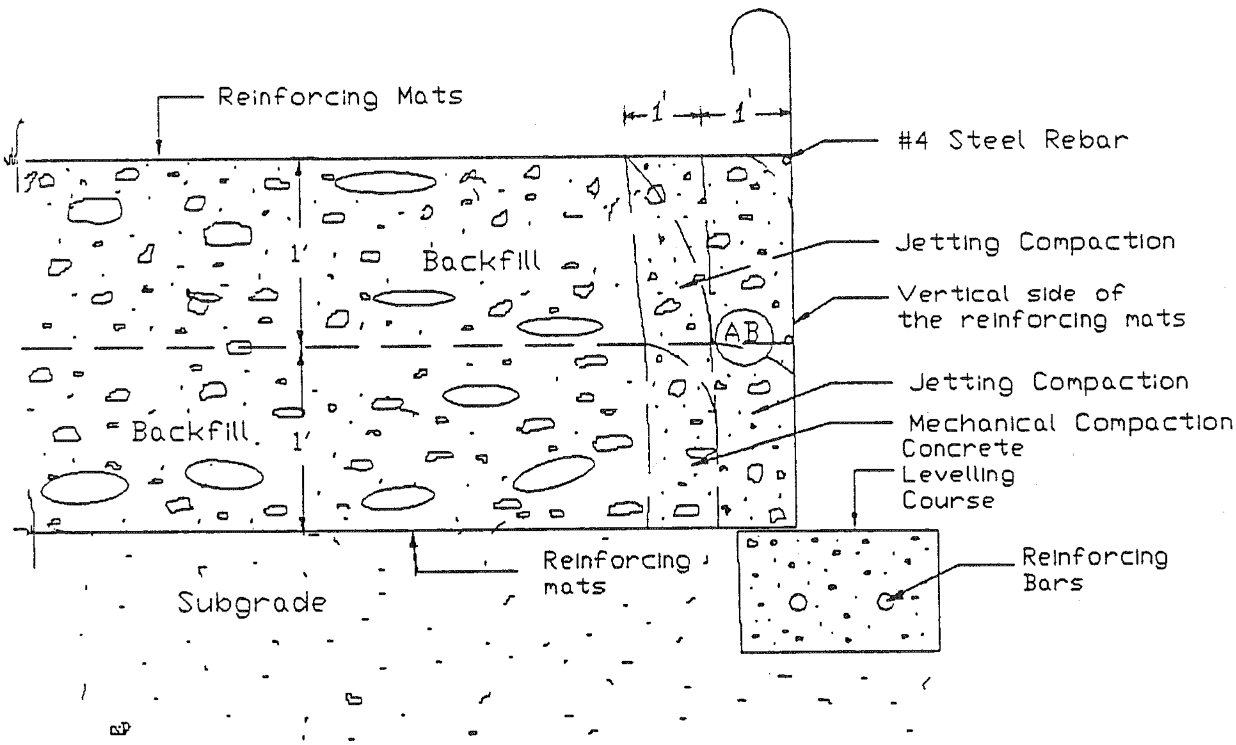


Figure 9 Schematic Diagram of the Lowest 2' Section of the Hilfiker Reinforced Embankment.

Figure 10 Dump Truck Used to Deliver Backfill Material and Hand Tampers Used for Compaction Near the Backing Mats.

Figure 11 Backfill Material Distributed on the Reinforcing Mats.

Attempts were made by the contractor to use only backfill material passing a 4" sieve in the 2' nearest the backing mats for the northeast and southwest walls. This was not required in the specifications, and quite often did not take place, as illustrated in Figure 12.

Figure 12 Backfill Material Larger Than a 4 " Sieve Placed Near the Backing Mats.

The backfill material used for the northeast and southwest walls was primarily river run obtained from the old embankments existing at the site. The material was processed on a grizzly and mixed. Although specifications required that all backfill material pass a 6" sieve (page D2), there were many rocks larger than that. Attempts were made to remove these rocks both manually and with a rock bucket. These attempts failed as a significant percentage of large rocks remained, as Figure 11 shows. Also, this material did not pass minimum resistivity requirements. Results of resistivity tests ranged from 536 ohm-cm to 2412 ohm-cm, whereas the special provisions of the project required 3000 ohm-cm minimum. As a result, the site was over-excavated by 1' prior to backfill placement, as required in the special provisions of the project. The approved material used to replace the over-excavated material was cohesionless and passed a 3" sieve. It was provided by Calmat of Arizona. The backfill for the southeast wall consisted entirely of the Calmat material. The backfill for all of the walls met specifications in regard to chloride content, sulfate content, and pH. APPENDIX F shows typical results of resistivity, pH, chloride, and sulfate tests for the on-site river run and the Calmat material. Random field tests on the density of the compacted backfill indicated 90 to 101 percent of the maximum dry density by AASHTO T-180. Specifications called for a minimum of 90 percent. Density test results are included as APPENDIX G.

The horizontal reinforcing mats were stacked on top of the backfill lifts and interlocked with the vertical backing mats. During construction the alignment of the embankment was kept vertical, checked with a 4' carpenter's level and string line. This is shown in Figure 13. Figure 14 shows the fully completed northeast wall embankment prior to the construction of the full-height concrete panels.

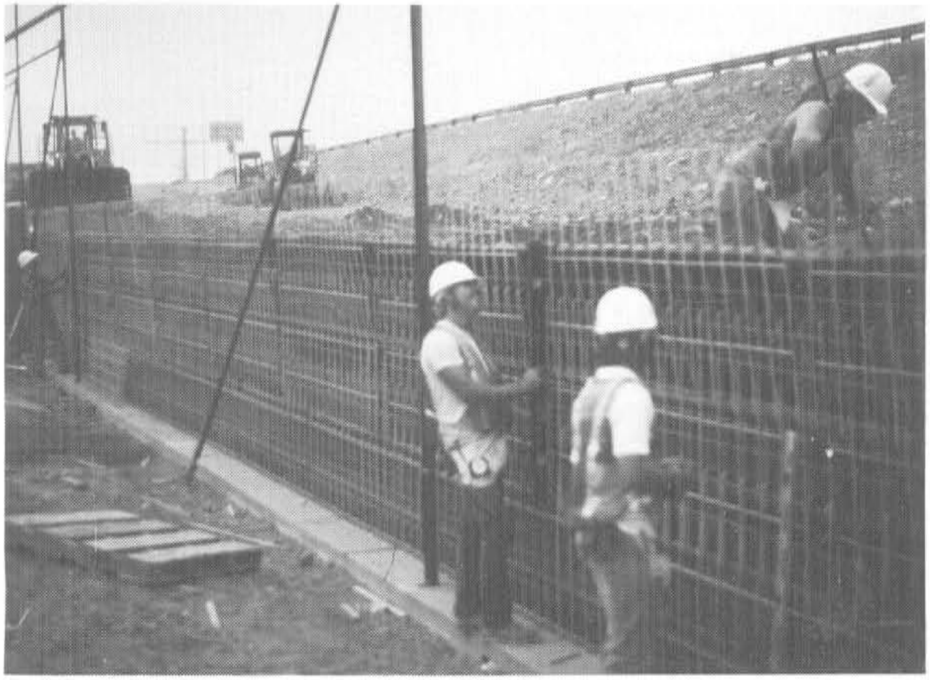


Figure 13 Workers Checking the Vertical and Horizontal Alignment of a Hilfiker Embankment.

Figure 14 NE Wall Embankment Prior to Construction of Full-Height Concrete Panels.

Panel Construction

Placement of the full-height concrete wall face followed the completion of the embankment. The procedure consisted of pouring the concrete from the top of the embankment down into full-height rubber inner forms and wood outer forms. The forms were configured to meet ADOT I-10 Project Architectural Rustication Detail M7 - A. Figure 15 is a photo of the rubber and wood forms used to attain the rustication.

Figure 15 Rubber and Wood Forms Used to Attain Specified Rustication.

The concrete was mixed at a batch plant and delivered to the site in transit mixers. The concrete was poured down the full face of the embankment, at times over 20'. Upon pouring, the concrete was vibrated with long internal vibrators as shown in Figure 16. After removing the forms, the concrete showed extensive honeycombing and air voids. Therefore, external vibrators were used in addition to the long internal vibrators. However, because the vibrations of the external vibrators did not transfer well through the rubber and wood forms, honeycombing and air voids were still visible. Use of external vibrators is shown in Figure 17. Figure 18 shows an example of the honeycombing and air pockets in the concrete panels.

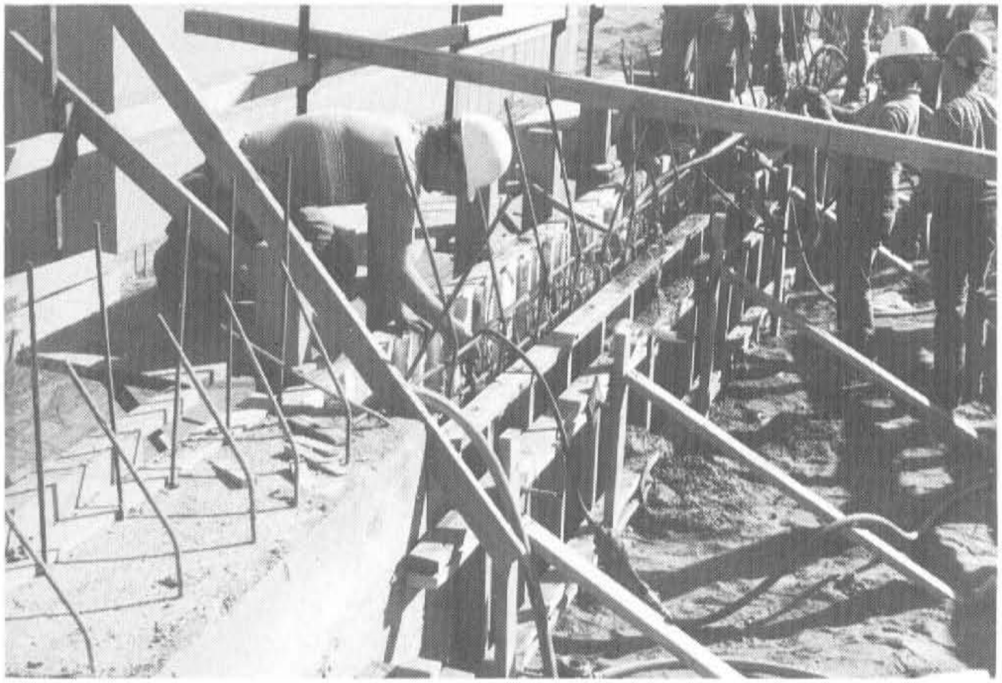


Figure 16 Long Internal Vibrators Initially Used to Vibrate the Concrete Panels.

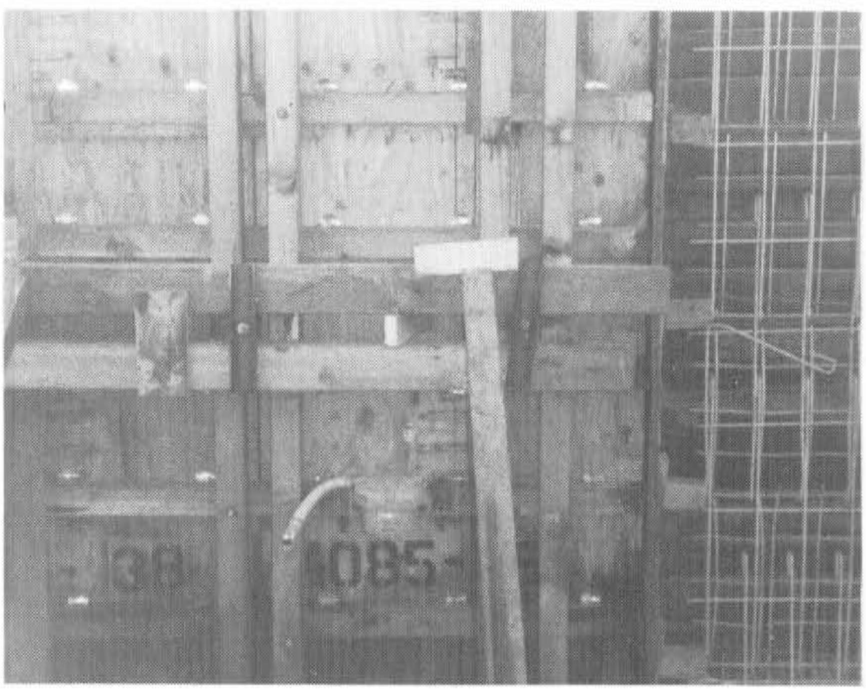


Figure 17 Additional External Vibrators Later Added to the Lower Sections of the Wall.

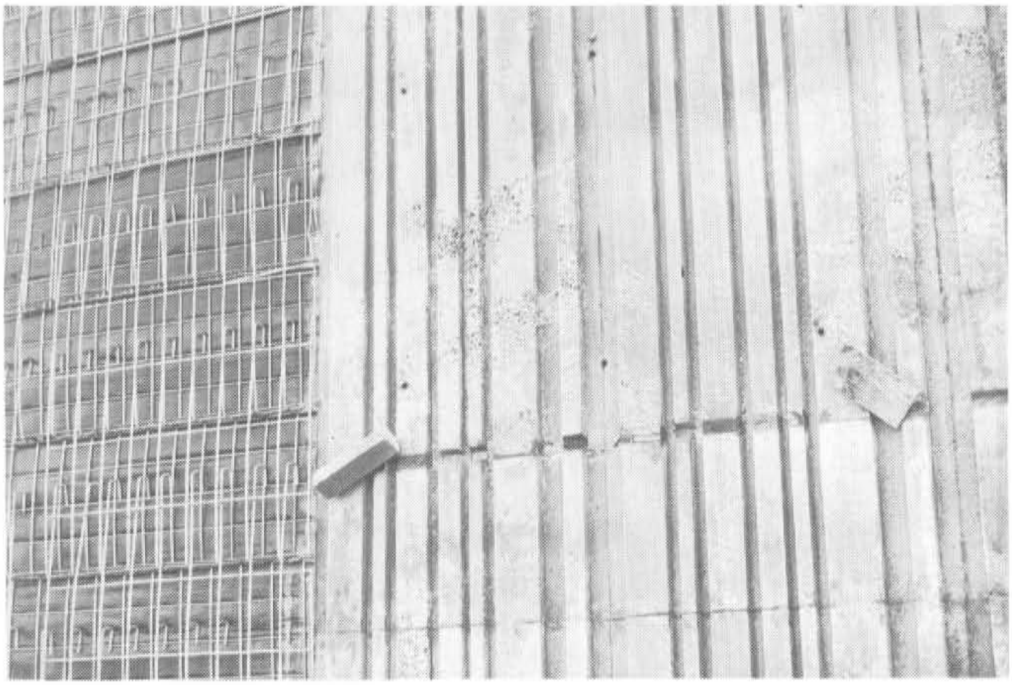


Figure 18 Honeycombing and Air Pockets Visible in the Full-Height Panels.

The wall was patched and painted; however, honeycombing and air pockets remained evident as shown in Figure 19.

Figure 19 Honeycombing Was Still Visible After Patching Attempts.

The contractor could not use tremie concrete nor larger diameter internal vibrators because of the limited clearance of the forms. The concrete panels were only 6" wide and had a maximum of 1.5" clearance for the internal vibrators. In an attempt to attain a higher quality finish, three different mix designs were tried by the contractor. APPENDIX H includes the different mix designs, and corresponding typical compression test results.

The Northeast Wall

Construction of the Hilfiker Reinforced Earth System of the northeast wall began in September, 1986. The construction of the wall was completed (minus painting) in January 1987. The wall ranged in height from 8' to 32', and was 1000' long. Construction notes specific to the northeast wall follow.

The river run backfill material did not pass resistivity requirements, and a $16,000 force account was initiated by ADOT to compensate the contractor for additional excavation. The subgrade was over-excavated 1' and filled with approved aggregate from Calmat of Arizona. The total backfill material volume was 15,270 cubic yards. Twenty field density tests were performed (APPENDIX G). The backfill met density specifications. All backfill material was specified to pass the 6" sieve, but this requirement was not always met.

The concrete wall panels originally were to have a 1:48 batter. However, a change order was initiated by the contractor to construct a vertical wall instead. The change order was approved by ADOT at no change in cost. An additional approved change order allowed the contractor to drop the concrete more than 8' without the use of approved pipes or tubes. The change order was requested because there was no clearance in the concrete panels to allow for the use of a tremie. There was no change in costs for this change order.

The plans did not provide for drainage outlets in the concrete panels. The panels immediately began to display honeycombing, air voids, and shrinkage and settlement cracks. The cracks were discontinuous and located primarily, but not limited to, the horizontal rustication of the wall. Figures 20 and 21 illustrate the horizontal cracks and air voids.

Additional work included removing the top reinforcing mat and backfill to accommodate the 15" needed for the concrete pavement (10" Portland Cement Concrete Pavement and 5" Lean Concrete Base). A design error provided only 6" clearance for the pavement. The work was performed under a $5000 force account. This force account also provided for the shortening of the lugs of the anchor slab to preclude them from interfering with the reinforcing mats. This is illustrated in a schematic included as Figure 22. The lugs were shortened from the standard 5' and 3.5', to 1'. Another force account required the contractor to seal the construction joint between the Hilfiker wall and the new PCCP. The contractor was provided $5000 to remove the existing filler and apply a bituminous joint sealant.

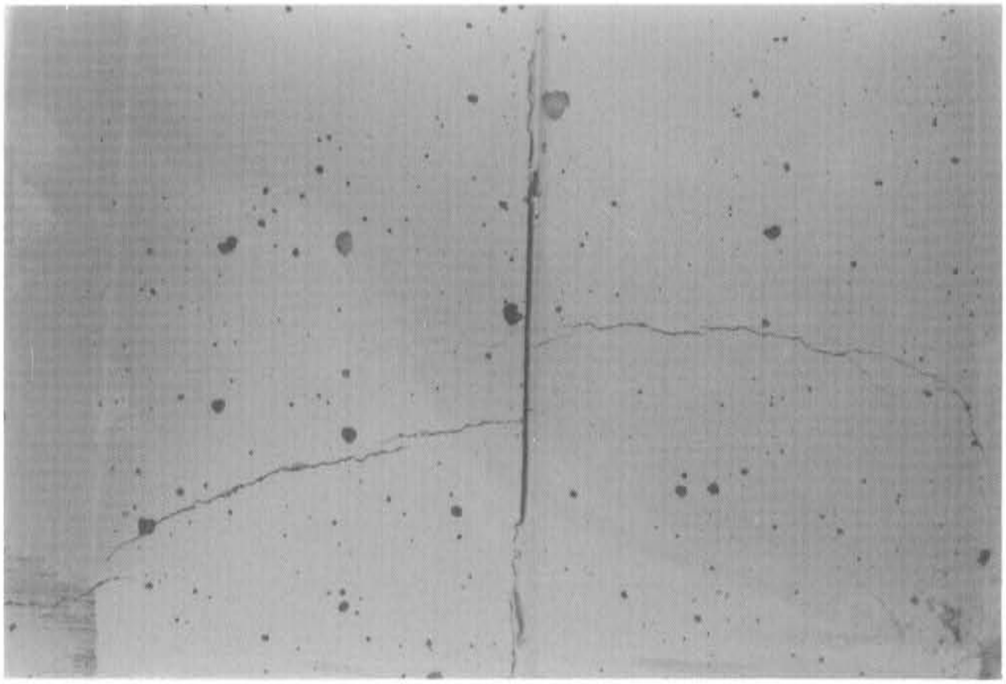


Figure 20 Hairline Shrinkage or Settlement Cracks and Air Voids.

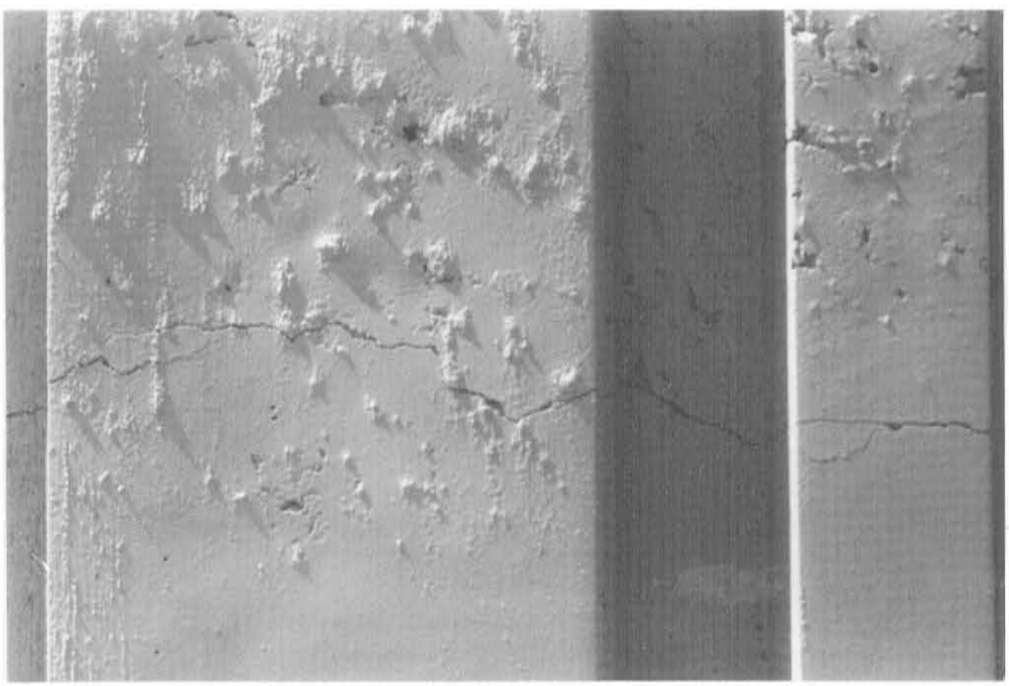


Figure 21 Air Voids and Horizontal Cracks in the Concrete Panels.

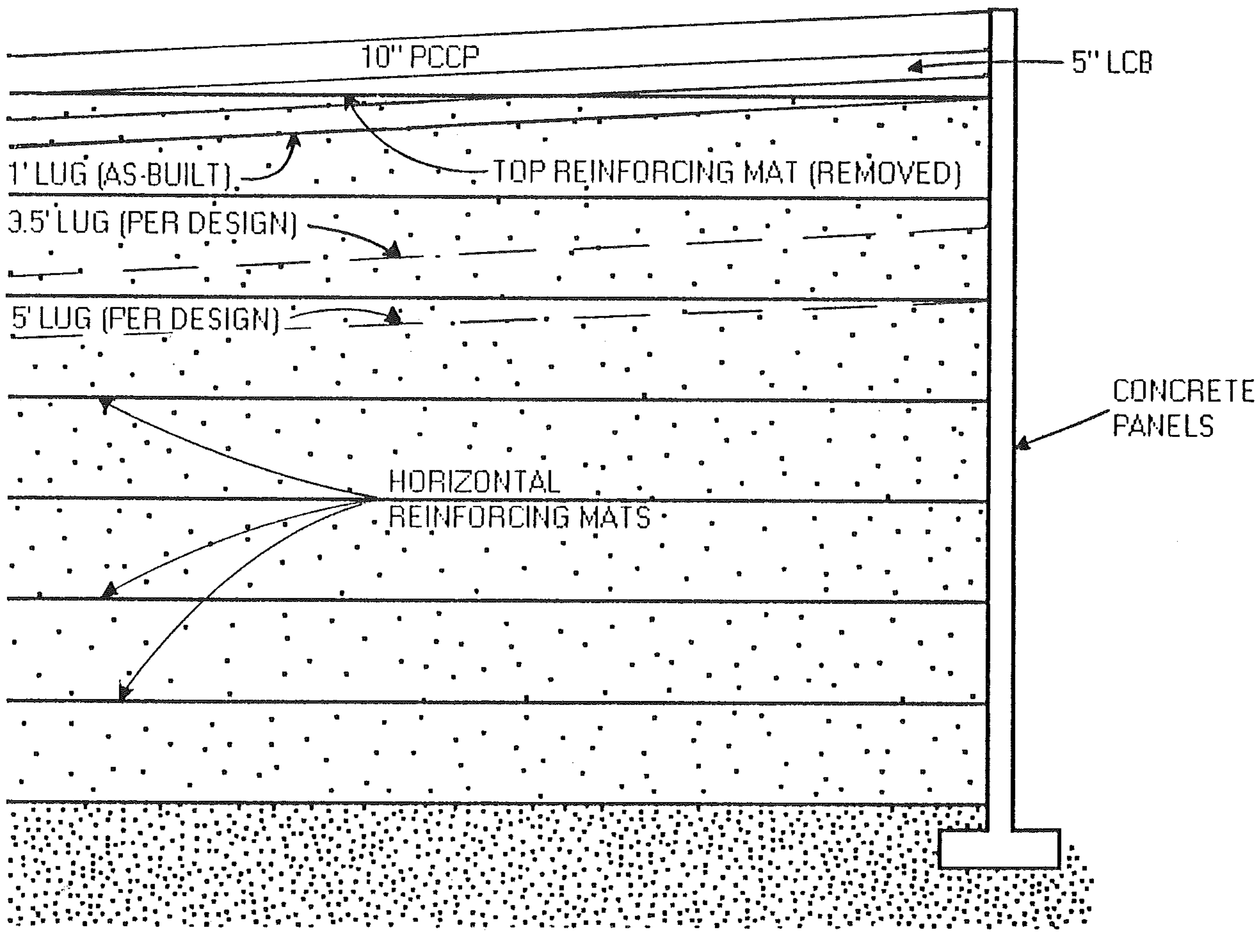


Figure 22 Schematic Diagram Illustrating the Conflict Between the top Reinforcing Mat and the Concrete Pavement (View is a Lengthwise Cross-Section).

Nine aluminum caps used as bench marks were embedded into epoxy at different locations along the concrete cap of the northeast wall. The caps were placed and surveyed to monitor the movement of the wall at future dates. The caps were initially surveyed by ADOT construction surveyors in March,1987.

The Southwest Wall

Construction of the Hilfiker Reinforced Earth System of the southwest wall began in October, 1986, and was completed in November, 1987. Construction was put on hold from March to September, 1987, as the contractor concentrated work on a different aspect of the overall project. The completed wall ranged in height from 4' to 30', and was 1220' long. Construction notes specific to the southwest wall follow.

The subgrade for the southwest wall needed to be over-excavated 8.5' to remove existing concrete rubble. A total of 22 field density tests were performed by ADOT during construction of the southwest wall (APPENDIX G). The backfill met density specifications. However, this backfill material did not meet gradation requirements. Project specifications called for all material to pass the 6" sieve. This was clearly not the case as a significant portion of the backfill was greater than 6". Figure 23 shows the varying size of aggregate.

Figure 23 Backfill Material With Rocks Larger Than the 6" Sieve.

The concrete panels of the face of the wall were poured similarly to the procedure followed for the northeast wall. Consistent with the other walls, there were no drainage outlets provided in the plans. Once again, cracks, honeycombing, and air voids were prevalent along the panels. Figure 24 shows patched honeycombed areas, and Figure 25 shows typical cracks ranging from 0.007" to 0.03" along the horizontal rustication of the concrete wall.

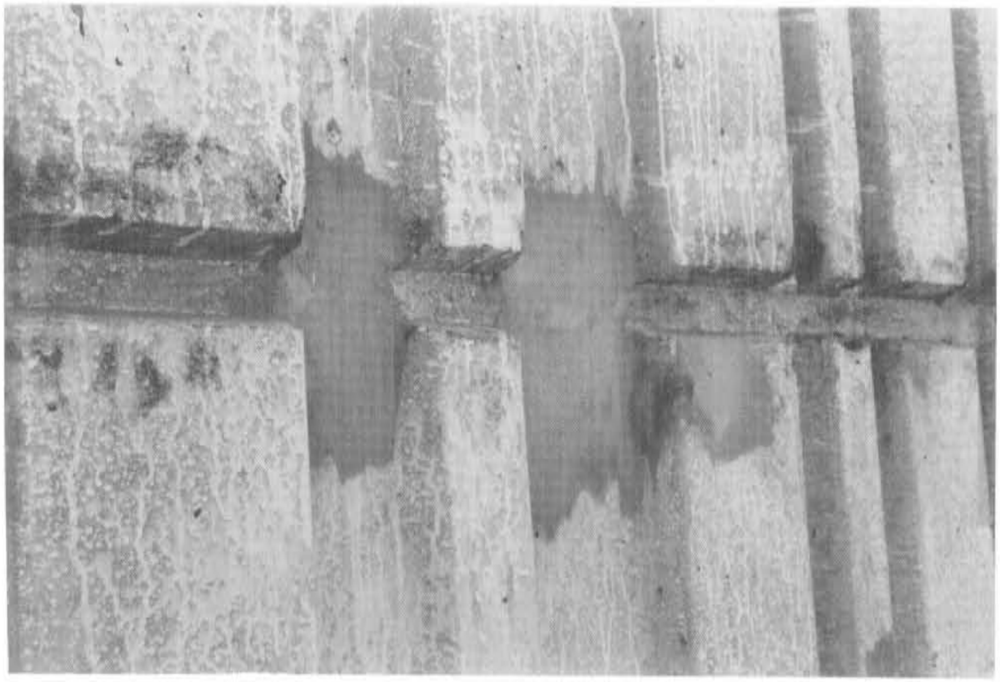


Figure 24 Patched Honeycombed Areas of the Southwest Wall.

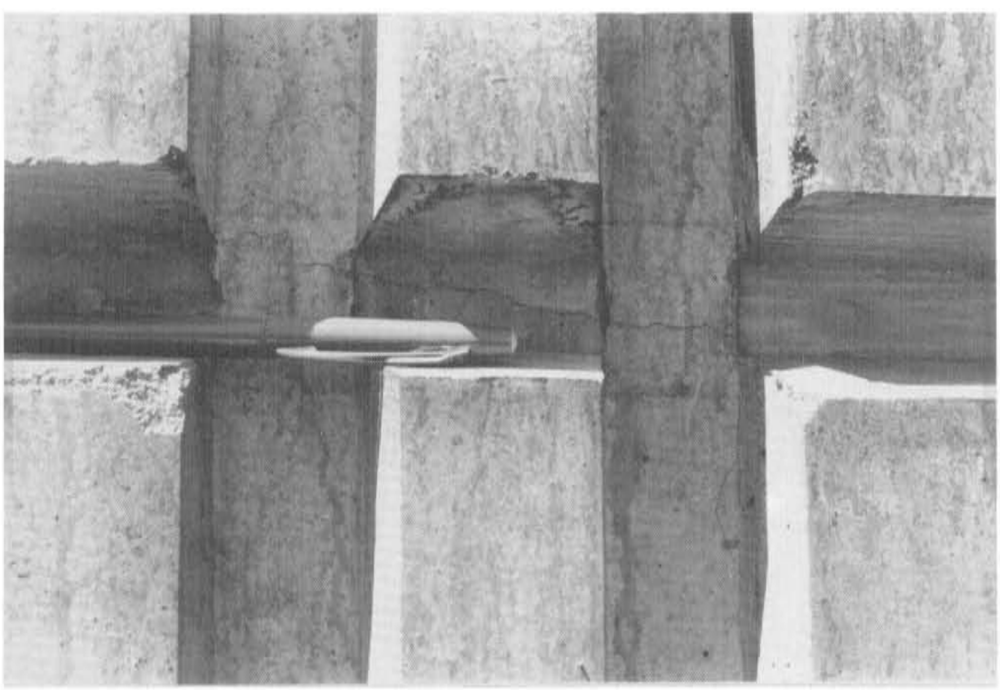


Figure 25 Typical Cracks Along the Horizontal Rustication of the Concrete Panels.

It is believed that these horizontal cracks exist in all of the horizontal rustication of the concrete walls. However, this was not verified at upper portions of the wall due to an inability to access these heights.

More severe voids and honeycombing, late in the construction of the southwest wall, resulted in the exposure of the embankment reinforcing steel as shown in Figures 26 and 27. In these instances, the contractor broke the face of the panels and replaced them with a higher quality concrete finish. Another remedy was extensive patching, shown in Figure 28.

Figure 26 Exposed Reinforcing Steel Due to Severe Honeycombing.

A problem developed at the interface of the bridge abutment wingwall and the Hilfiker concrete panels. Prior to the completion of the last panel of the wingwall, a significant amount of the backfill washed out due to heavy rainfall. Figure 29 shows the backfill material that had been lost through the abutment. Figure 30 is the crevice formed by the seepage. Figure 31 is the void created at the top of the embankment, noticeable because the PCCP had not yet been placed.

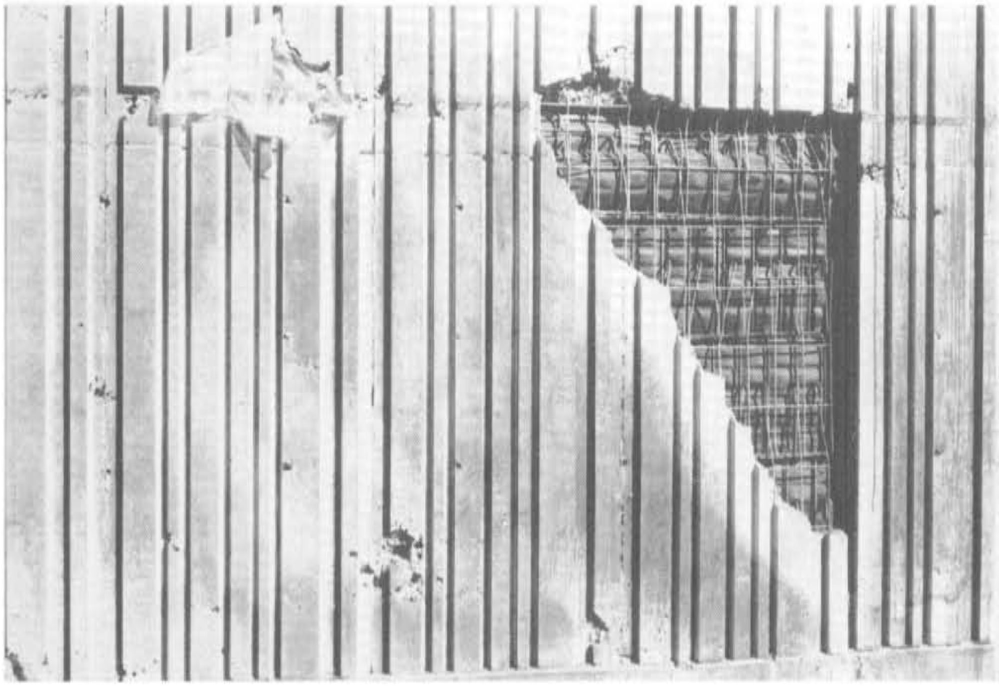


Figure 27 Two Panels That Needed to be Broken Out and Replaced.

Figure 28 Severe Honeycombing Treated by Patching.

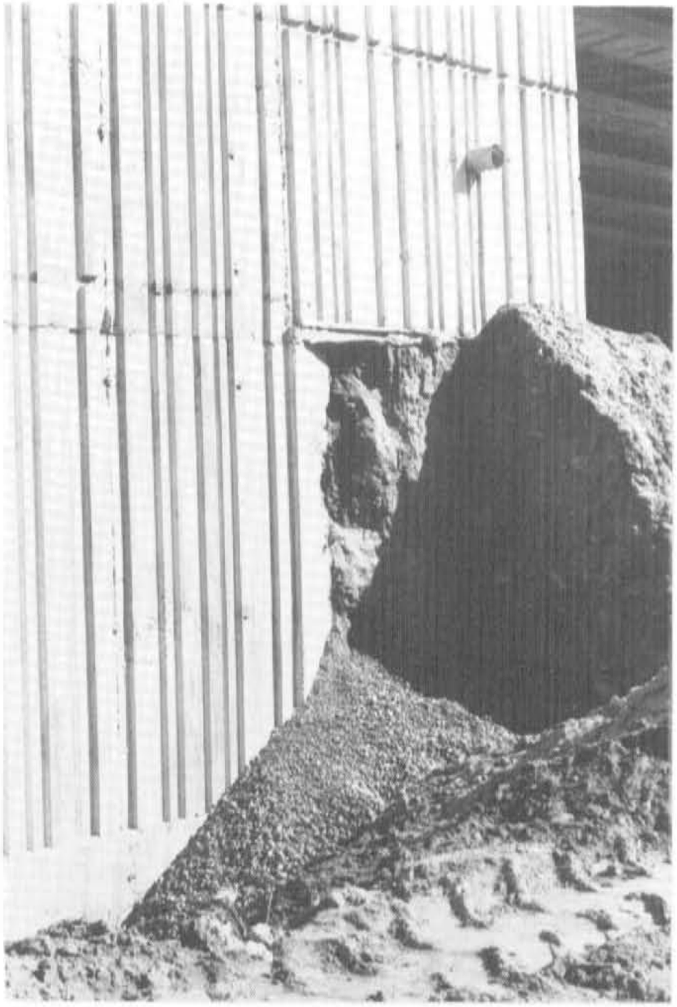


Figure 29 Backfill Material that Seeped Through Abutment Wingwall.

Figure 30 Crevice Formed by Washout of Backfill Material.

Figure 31 Void at the Top of the Embankment Due to Loss of Backfill Material.

To remedy this situation, the contractor supported the bottom of the abutment wingwall as shown in Figure 32, and filled the void with a structural backfill material. The material was then compacted with a jetting technique.

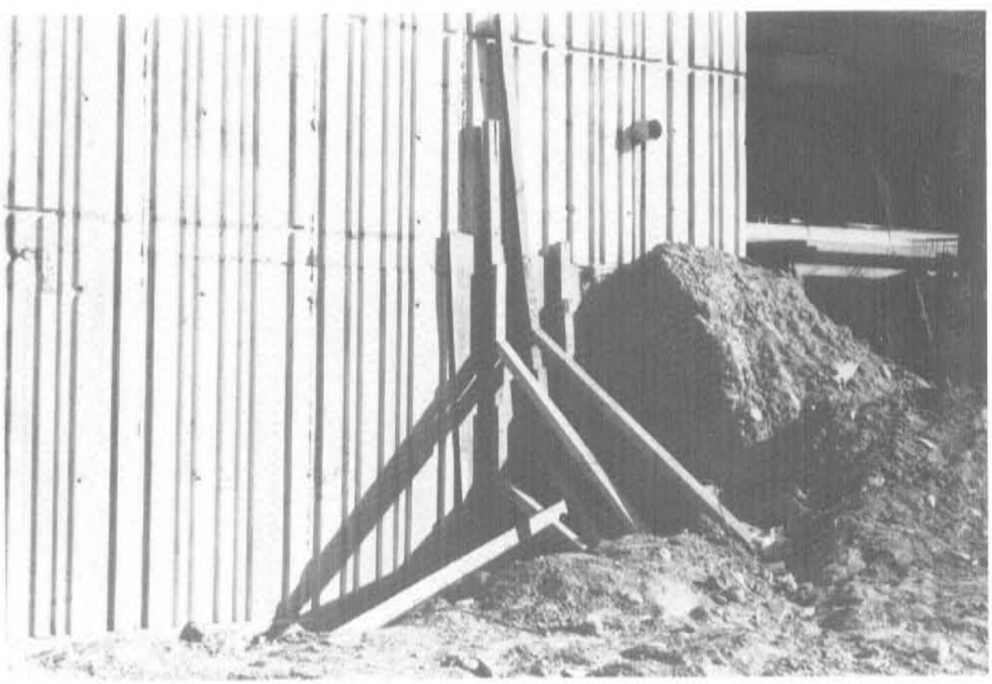


Figure 32 Support to Bridge Abutment for Refilling Rain Caused Crevice.

During the placement of the forms for the concrete panels, a crane overturned and damaged the wall's steel reinforcement. Figures 33 and 34 show the overturned crane and the resulting damage. Apparently the operator failed to extend the crane's outriggers prior to lifting some forms. The damage was repaired by straightening the steel and placing additional rebar. Figure 35 shows the site of the damaged section after repairs had been completed.

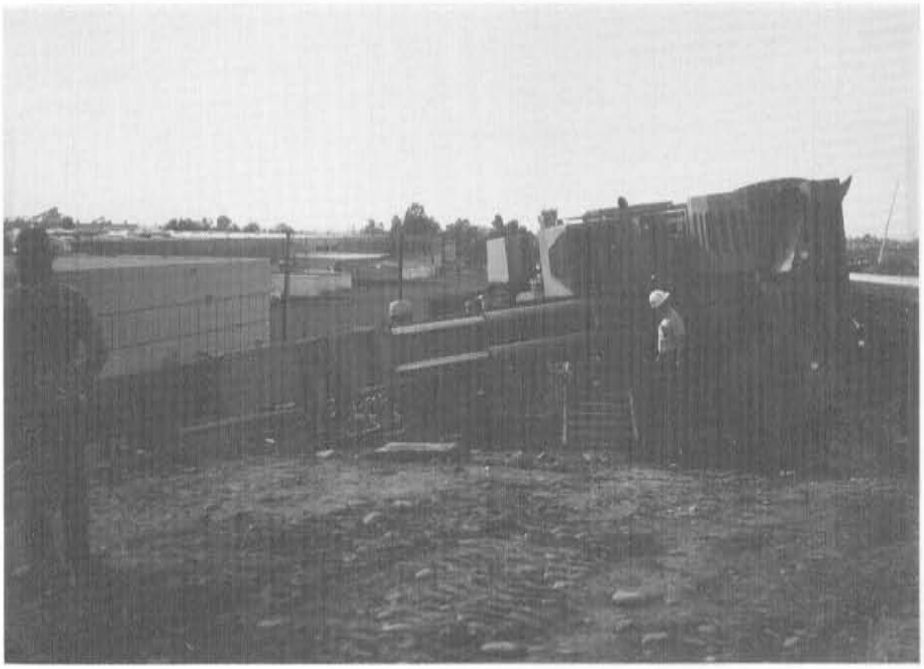


Figure 33 Overturned Crane on SW Embankment.

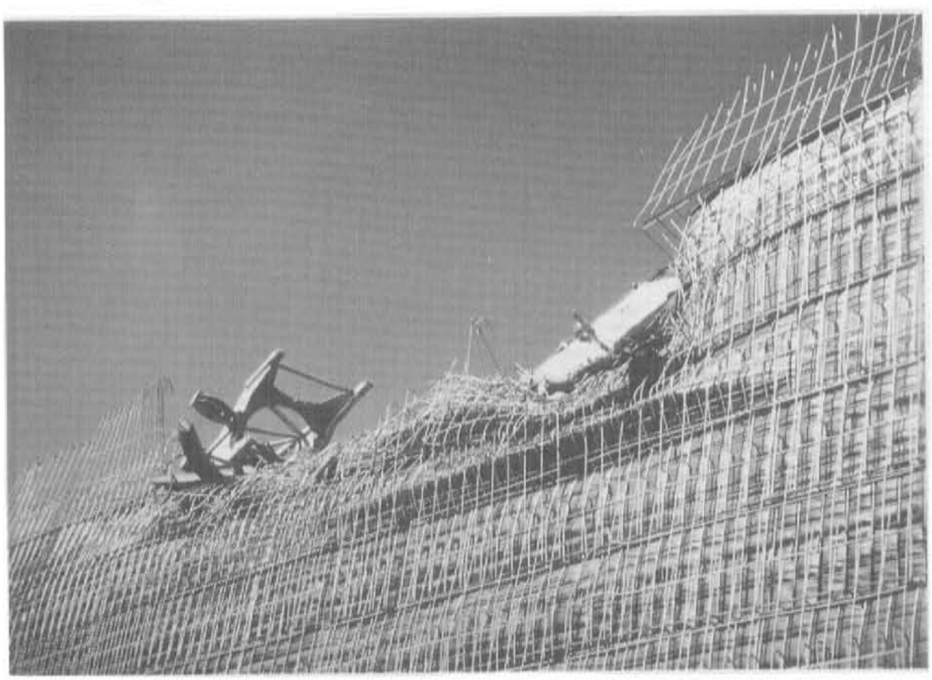


Figure 34 Damage Caused By Overturned Crane.

Figure 35 Site of Damaged SW Wall Section After Repair.

Change orders were requested by the contractor to avoid cutting the reinforcing mats of the embankment. The first change order was necessary because some of the catch basins in the plans interfered with the reinforcement of the embankment. This change order allowed for the use of catch basin blockouts, and a reduction in the slope of a drainage pipe from the catch basins. The cost of the change order was $955. The second change order was initiated because of underground features of the guardrail and barrier transitions in the plans. In order to avoid the mats, the barrier was extended beyond the extent of the reinforcement. A force account was established to blockout the foundation of a light pole that was in conflict with the reinforcing mats. The force account paid $1500 to the contractor.

Upon completion of the southwest wall, 15 aluminum caps were placed in epoxy along the length of the wall. The caps were surveyed by ADOT personnel in December, 1987, to be used to monitor movements of the wall at future dates.

Figures 36 and 37 show the southwest Hilfiker wall at completion.

Figure 36 Completed SW Wall (Photo From the West End).

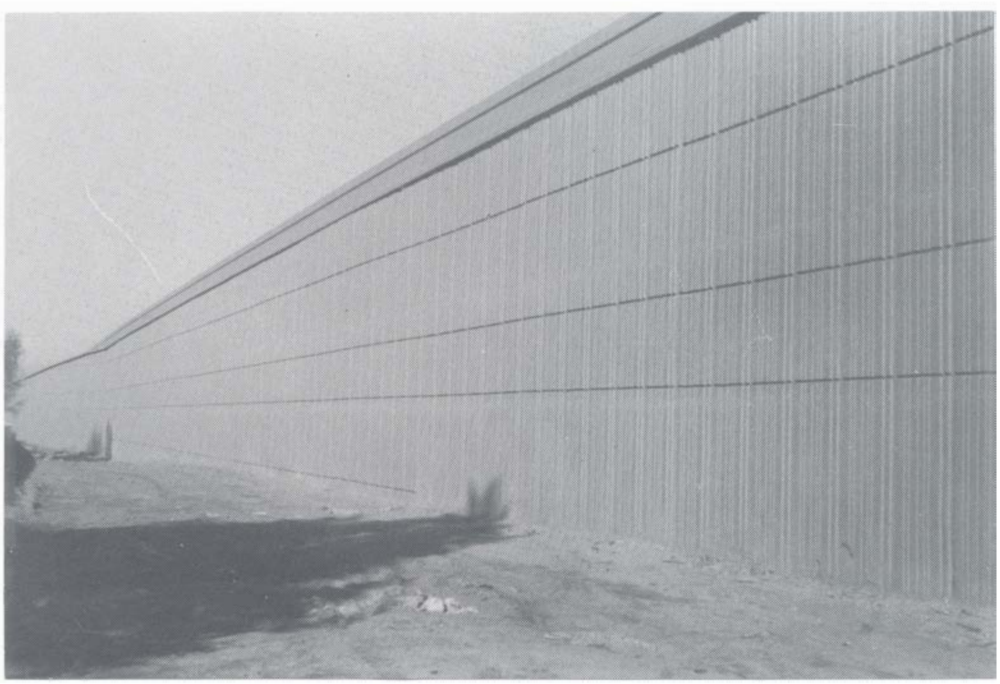


Figure 37 Completed SW Hilfiker Wall (Photo Facing Northwest).

The Southeast Wall

Construction of the Hilfiker Reinforced Earth System of the southeast wall began in August, and was completed in December, 1987. The completed wall ranged in height from 3' to 12', and was 680' long. Construction notes specific to the southeast wall follow.

The backfill material for the southeast wall consisted entirely of material supplied by Calmat of Arizona. The material passed resistivity, pH, and chloride requirements, and all material passed the 3" sieve. Twelve density tests were performed by ADOT during construction of the southeast wall (APPENDIX G), and the backfill passed density requirements.

Towards the end of the embankment construction, it became evident that the contractor did not have enough reinforcing mats to raise the embankment to plan elevation. Upon approval from the Hilfiker company, the contractor increased the space between mats to 3' rather than 2.5' as required by the plans. Figure 38 shows the completed embankment prior to placing the concrete forms.

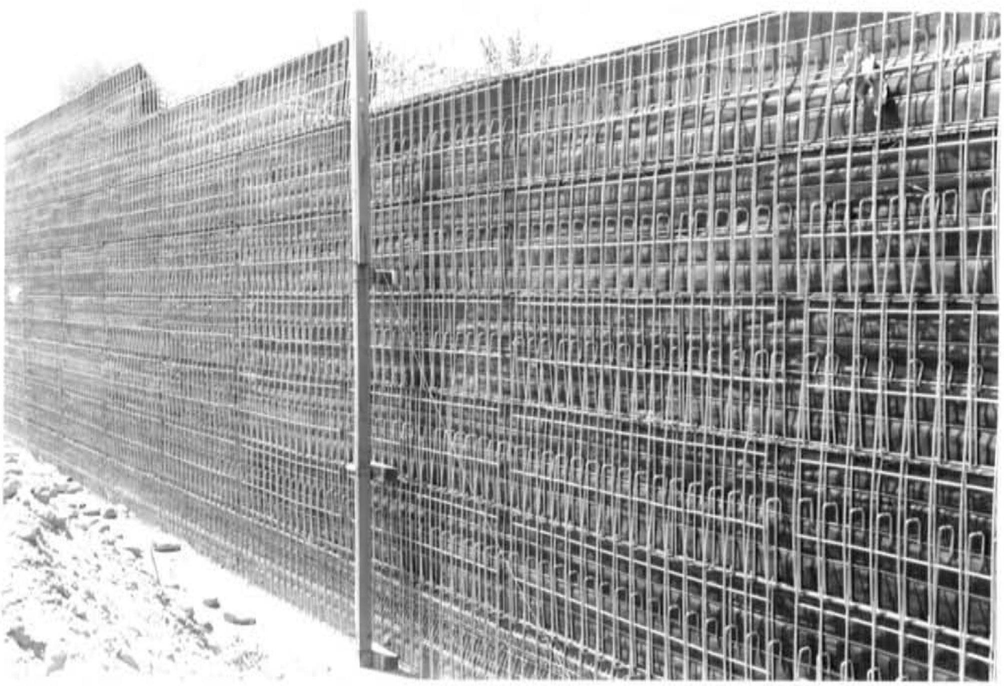


Figure 38 SE Embankment Prior to Placement of the Concrete Forms.

The concrete for the panels of the face of the wall was poured in similar fashion to the other walls. Figure 39 shows the forms in place. The southeast wall panels, like the others, had no drainage outlets specified in the plans. However, this concrete wall had noticeably fewer shrinkage cracks, and a better quality finish than the other walls. This was probably due to the relative height and length of this wall.

Figure 39 Forms of the SE Wall Were Placed Similarly to the Other Walls.

Upon the completion of construction, 9 aluminum caps were embedded in epoxy along the top of the southeast wall. The caps were surveyed to monitor movements of the wall at future dates. Figure 40 is a photograph of the completed southeast Hilfiker wall.

Figure 40 Completed SE Hilfiker Wall (Photograph Facing East).

COST

The lump sum cost for the construction of the three retaining walls was \$1,150,000, as bid for the project. The contractor indicated that this amount was not sufficient to cover the construction costs of the walls, but did not comment on the actual figures. Dividing the lump sum \$1,150,000 by the construction area (68,862 ft^2) results in a cost of \$16.70 / ft^2 paid by ADOT.

EVALUATION

Evaluation of the Hilfiker Reinforced Earth Systems was conducted in the form of visual inspection of the walls, and surveys of the aluminum caps epoxied to the walls to monitor movement. The evaluation period extended from completion of the walls through September, 1991.

Surveys

A final survey of the aluminum caps embedded in epoxy on each of the three Hilfiker walls was performed during July and August, 1991. The survey consisted of determining the location and elevation of the aluminum caps, and comparing them with the results of the surveys performed upon completion of the walls. The overall movement of the caps on each of the three walls is given in Tables 2, 3, and 4. Figures 41, 42, and 43 are maps of the aluminum caps of each of the walls, with arrows indicating the direction and magnitude of movements.

Cap Number	Elevation Change (ft.)	Horizontal Movement (ft.)
1	-.0033	.1424
2	+.0007	.0968
3	-.0233	.0443
4	-.0293	.0292
5	-.0353	.0496
6	-.0393	.0256
7	-.0423	.0652
8	+.0403	.0442
9	-.0613	.0183
Maximum	-.0613 (Cap 9)	.1424 (Cap 1)

Movements based on surveys 3/13/87 and 7/20/91.

Table 2 Movements of the NE Hilfiker Wall.

Cap Number	Elevation Change (ft.)	Horizontal Movement (ft.)
1	-.0340	.1261
2	-.0300	.0786
3	-.0240	.0915
4	-.0050	.0790
5	-.0040	.0673
6	-.0150	.0253
7	-.0140	.0337
8	-.0280	.0512
9	-.0290	.0390
10	-.0360	.0861
11	-.0230	.0734
12	-.0080	.1846
13	-.0100	.2559
14	+.0080	.3101
15	.0000	.2544
Maximum	-.0360 (Cap 10)	.3101 (Cap 14)

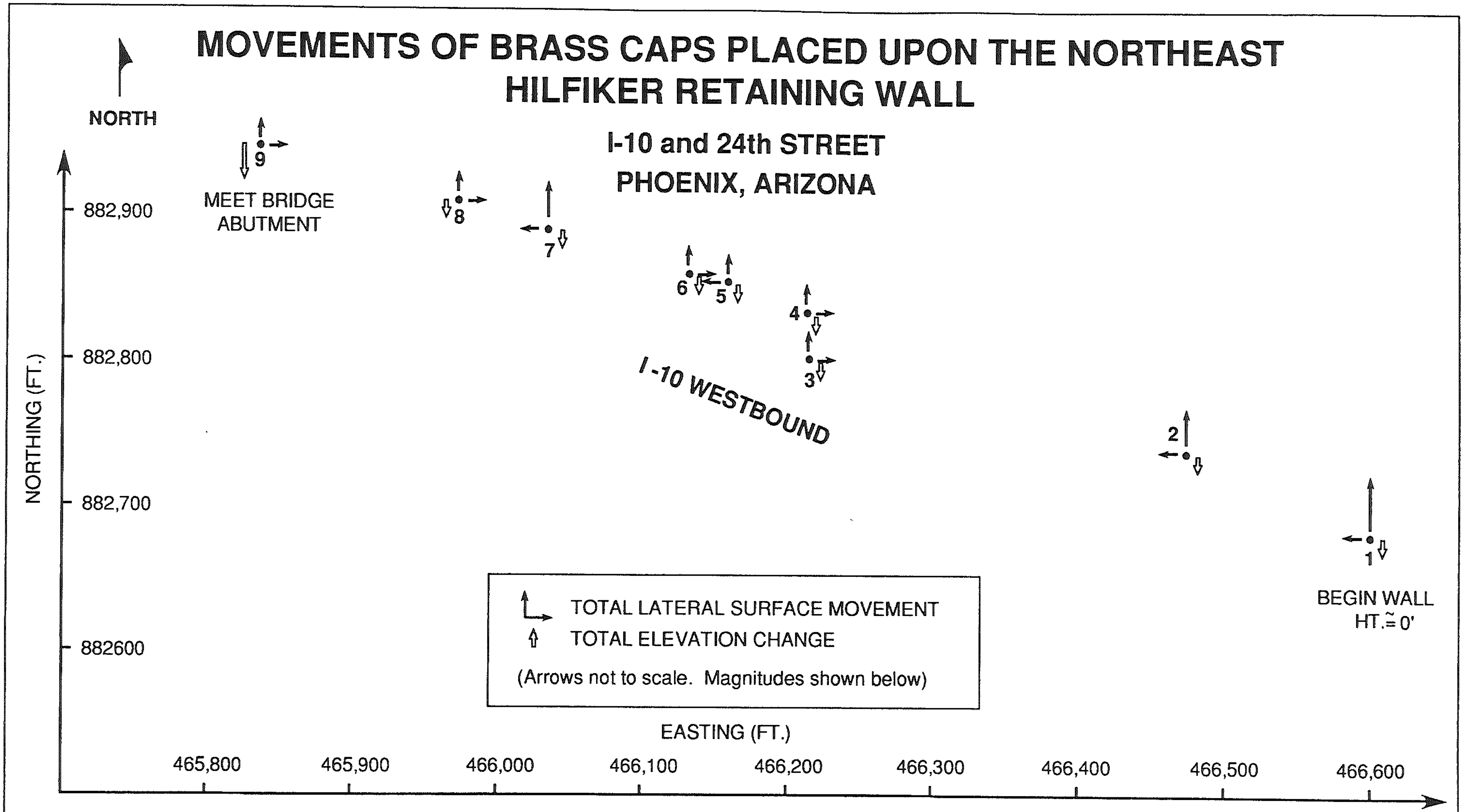
Movements between surveys 12/15/87 and 8/03/91.

Table 3 Movements of the SW Hilfiker Wall.

Cap Number	Elevation Movement (ft.)	Horizontal Movement (ft.)
1	N/A	N/A
2	-.0800	.1362
3	-.0780	.0472
4	-.0770	.1335
5	-.0730	.0536
6	-.0730	.0874
7	-.0630	.1622
8	-.0550	.1231
9	-.0520	.1958
Maximum	-.0800 (Cap 1)	.1958 (Cap 9)

Movements Between Surveys 1/13/88 and 8/3/91. All aluminum caps have been removed. Movements are based on surveyor's estimate of position of caps for 8/3/91 survey.

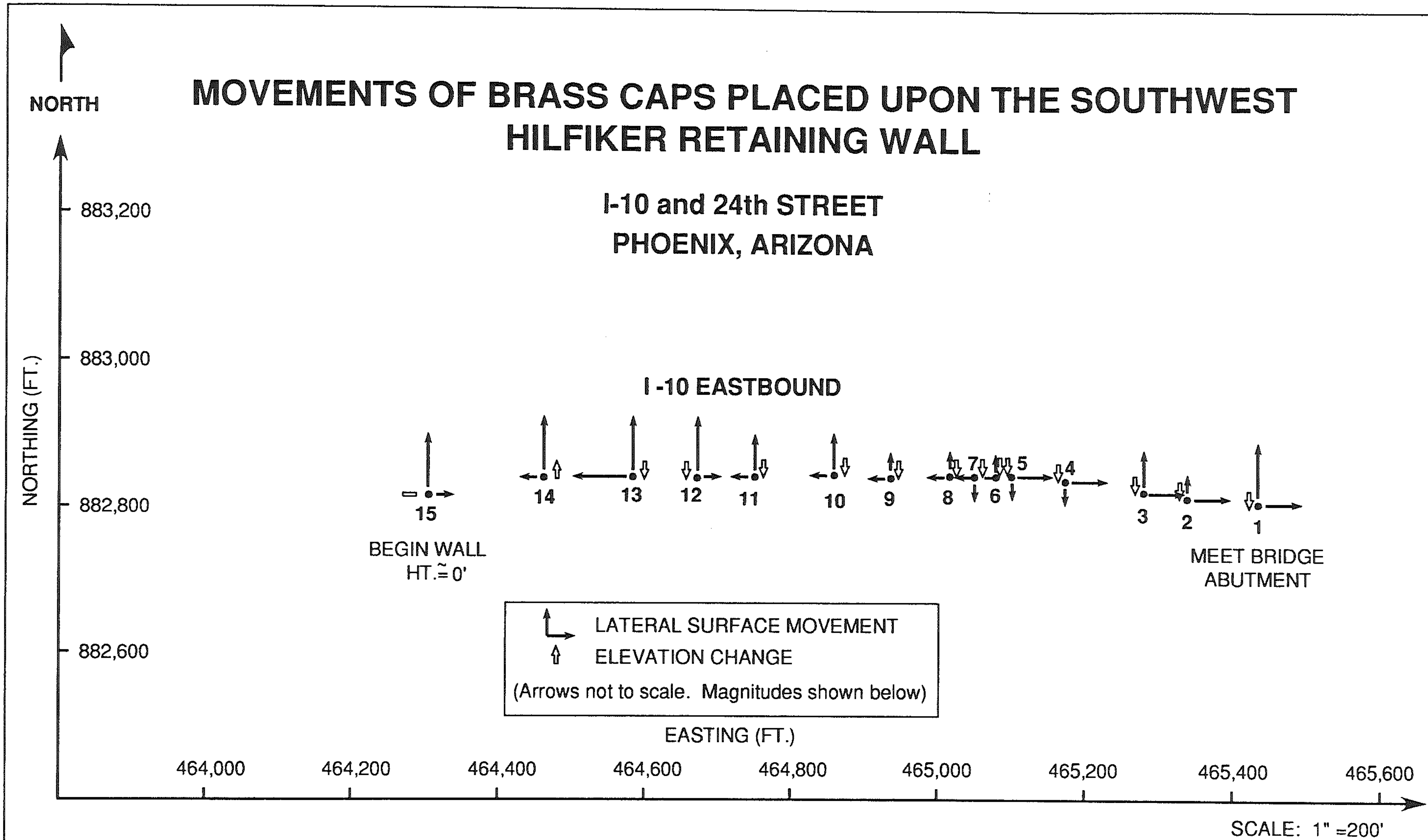
Table 4 Estimated Movements of the SE Hilfiker Wall.



	CAP #	9	8	7	6	5	4	3	2	1
TOTAL MOVEMENT	North (ft.)	.0061	.0363	.0644	.0242	.0421	.0285	.0443	.0943	.1415
	East (ft.)	.0173	.0252	-.0101	.0082	-.0263	.0065	.0007	-.0217	.0157
	Elevation(ft.)	-.0613	-.0403	-.0423	-.0393	-.0353	-.0293	-.0233	.0007	-.0033

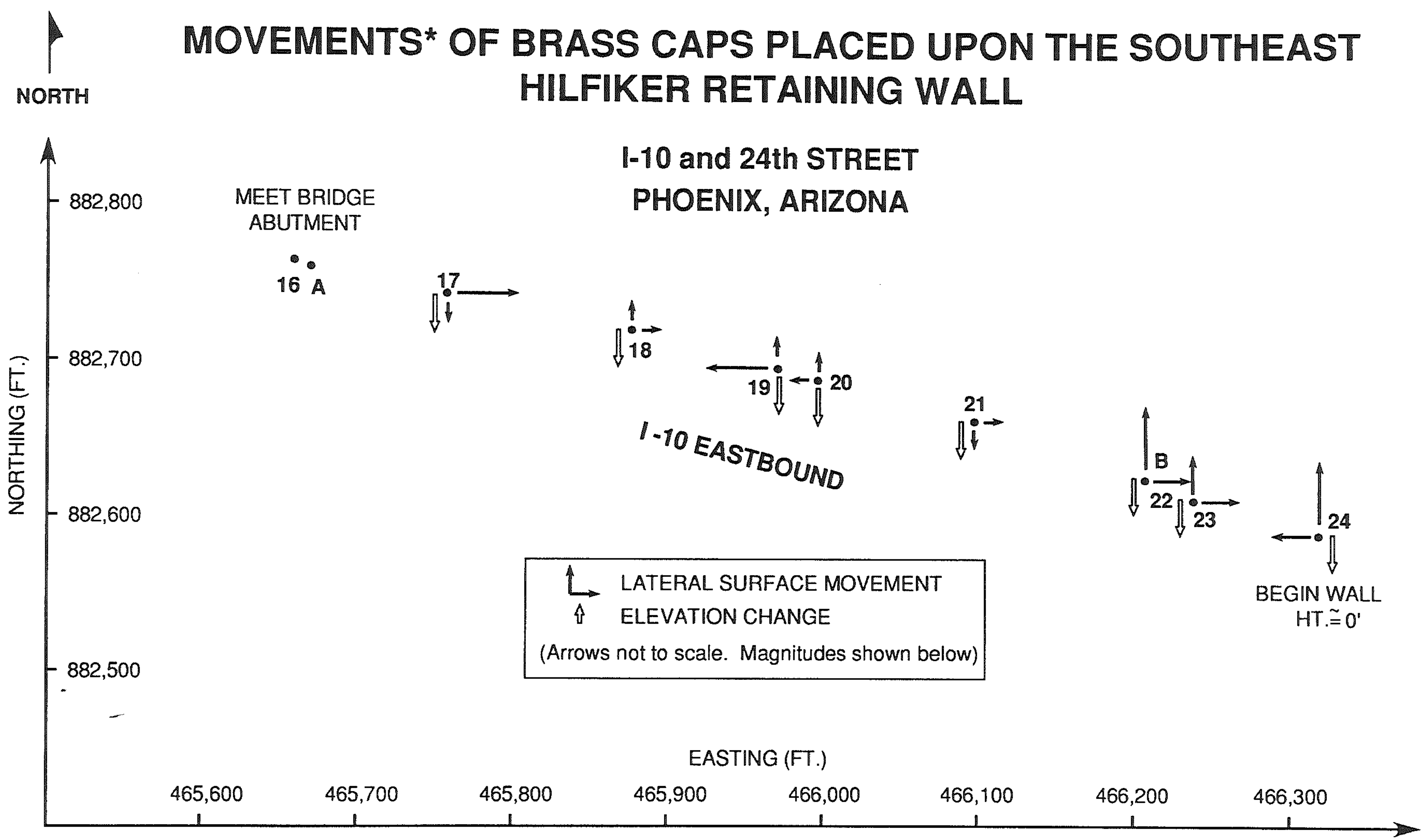
SCALE: 1" = 100'

Figure 41 Movements of the Caps Placed on the Northeast Wall.



CAP #	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1
ΔNorth (ft.)	.2542	.3081	.2251	.1844	.0721	.0762	.0360	.0215	-.0198	.0011	-.0189	-.0410	.0687	.0493	.1124
ΔEast (ft.)	.0110	-.0352	-.1218	.0088	-.0140	.0400	-.0150	-.0465	-.0273	-.0253	.0646	.0675	.0605	.0612	.0571
ΔElevation(ft.)	0	+.008	-.010	-.008	-.023	-.036	-.029	-.028	-.014	-.015	-.004	-.005	-.024	-.030	-.034

Figure 42 Movements of the Caps Placed on the Southwest Wall.



CAP #	1	2	3	4	5	6	7	8	9	A	B
ΔNorth (ft.)	–	-.0265	.0387	.0275	.0218	-.0643	.1494	.0850	.1872	–	–
ΔEast (ft.)	–	.1336	.0271	-.1306	-.0490	.0592	.0634	.0891	-.0574	–	–
ΔElevation(ft.)	–	-.08	-.078	-.077	-.073	-.073	-.063	-.055	-.052	–	–

SCALE: 1" = 100'

*MOVEMENTS CANNOT BE TAKEN AS PRECISE; SURVEY CAPS HAVE BEEN REMOVED. DATA BASED ON SURVEYORS ESTIMATE.

Figure 43 Movements of the Caps Placed on the Southeast Wall.

There is reason to doubt the reliability of the results of the surveys. Notice, for example, the northeast wall. Table 1 indicates that the aluminum cap with the greatest lateral movement, and a significant change in elevation is Cap 1. Figure 41 shows that Cap 1 is at the low end of the wall, and it would be expected that this cap would be the least susceptible to movement.

The aluminum caps embedded in epoxy on the southeast wall have been removed. Apparently they were scraped off by some sort of heavy vehicle, although this would be difficult considering the caps proximity to the concrete barriers placed on top of the wall. Figure 44 shows the location of a removed aluminum cap.

Figure 44 Aluminum Caps of the SE Wall Have Been Removed.

Additionally, the southeast wall has two aluminum caps that have not been surveyed. These caps are similar to the caps originally placed on the walls. Neither the ADOT surveyors who performed the initial and final surveys of the walls, nor ATRC personnel, are aware of the origin of these caps.

Cracks

There is a large (0.5" to 1.5") vertical crack at the interface between the southeast Hilfiker wall and the southeast abutment of the I-10 and 24th Street bridge. The retaining wall has settled about an inch relative to the bridge abutment at this point. Figure 45 is a photo of this crack along the face of the wall, and Figure 46 shows the damage to the concrete at the top of the wall at the crack.

Figure 45 Vertical Crack at the Interface of the SE Wall and the Bridge Abutment.

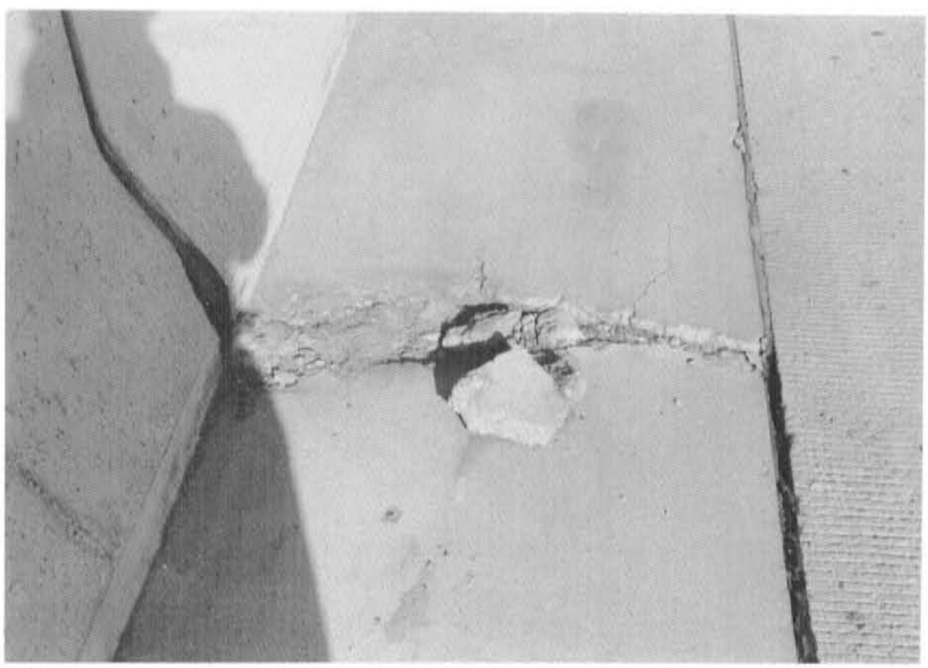


Figure 46 Damage to the Concrete at the Top of the Cracked Interface.

A large erosion channel had formed immediately below the vertical crack, yielding evidence that water was entering the reinforced earth embankment and exiting through this crack. Following rains it was noticed that sediment was carried into 24th Street. Figure 47 shows the erosion channel, which has at times been filled in by landscape maintenance crews.

Figure 47 Erosion Channel Immediately Below Vertical Crack of the SE Wall.

The two other Hilfiker walls were subject to similar phenomena; vertical cracks at the bridge abutment interface. However, the cracks were not as severe. Below the southwest wall crack there was a small erosion channel. There was not a channel below the northeast wall crack. Figures 48 and 49 show the Hilfiker wall - bridge abutment vertical cracks for the southwest and the northeast walls. Many hairline vertical and horizontal cracks are visible throughout all three of the retaining walls at this interchange.

Deflection of Slabs

In October, 1991, Local ADOT construction personnel reported that the anchor slab was deflecting relative to the approach slab of the southeast wall when 18-wheel trucks passed. Further inspection showed that this deflection was also evident, to a lesser extent, on the southwest wall. The deflections were not measured, but it appeared as though the maximum deflection on the southeast wall was about an inch.

Figure 48 Interface of SW Hilfiker Wall and the 24th Street Bridge Abutment.

Figure 49 Interface of NE Hilfiker Wall and the 24th Street Bridge Abutment.

Other Retaining Walls

In addition to the Hilfiker retaining wall Experimental Project at I-10 and 24th Street, the ATRC has been monitoring three additional full-height retaining walls at the request of the FHWA. Those retaining wall systems are included in Table 5.

Project Number	Description	System Design
IR-10-3(224)	Moreland St. Drop Structure	Hilfiker/Gravity
IR-10-3(192)	Warner Road I.C.	Hilfiker
I-10-3(220)	SPRR Overpass	VSL

Table 5 Additional Retaining Wall Projects Monitored at the Request of the FHWA.

Moreland Drop Structure

The Moreland Street Drop Structure is located in Phoenix between the ramp from westbound I-10 to northbound SR-51 and the ramp from westbound I-10 to eastbound Loop 202. Around the drop structure are two full-height retaining walls, ranging in height from 0' to approximately 13'. The western wall is part of a Hilfiker Reinforced Soil Embankment System, and was constructed with the drop structure in 1986. The southern wall is a gravity retaining wall, completed in 1988. Figure 50 depicts the geometry of the site, and Figure 51 is a photograph of the walls taken facing north. The walls can be differentiated at the site by their concrete caps. The Hilfiker wall is 10.5" across the top, whereas the gravity wall is 16" across the top.

Inspection of this facility included mapping the vertical cracks, expansion joints, and weep holes of both sections of the wall, and recording the widths of the vertical cracks. The cracks varied in width from 0.007" to 0.1" with the exception of a single large vertical crack along the west face of the Hilfiker wall. The crack is 0.5" wide; however, there is no vertical displacement across the crack at the top of the wall. Figure 52 is a photo of this large vertical crack. APPENDIX I is a record of the location and size of cracks, expansion joints, and weep holes along both the gravity wall and the Hilfiker Wall.

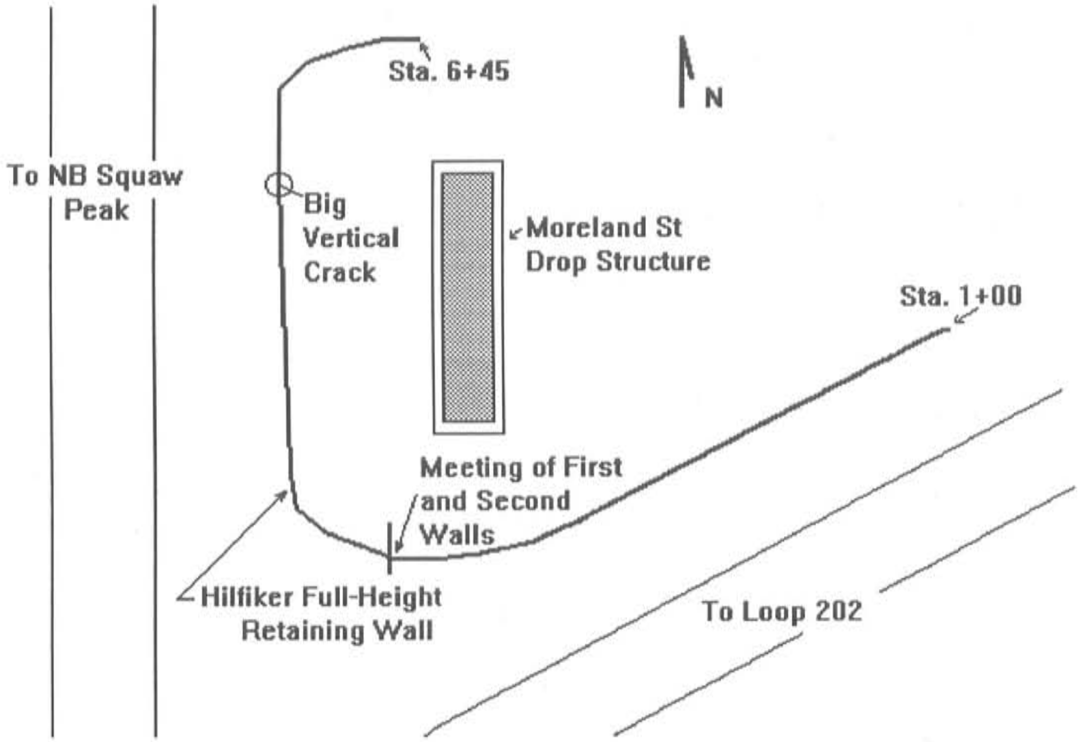


Figure 50 Moreland Street Dropout and Retaining Walls.

Figure 51 Retaining Walls About the Moreland Street Drop Structure (Looking North).

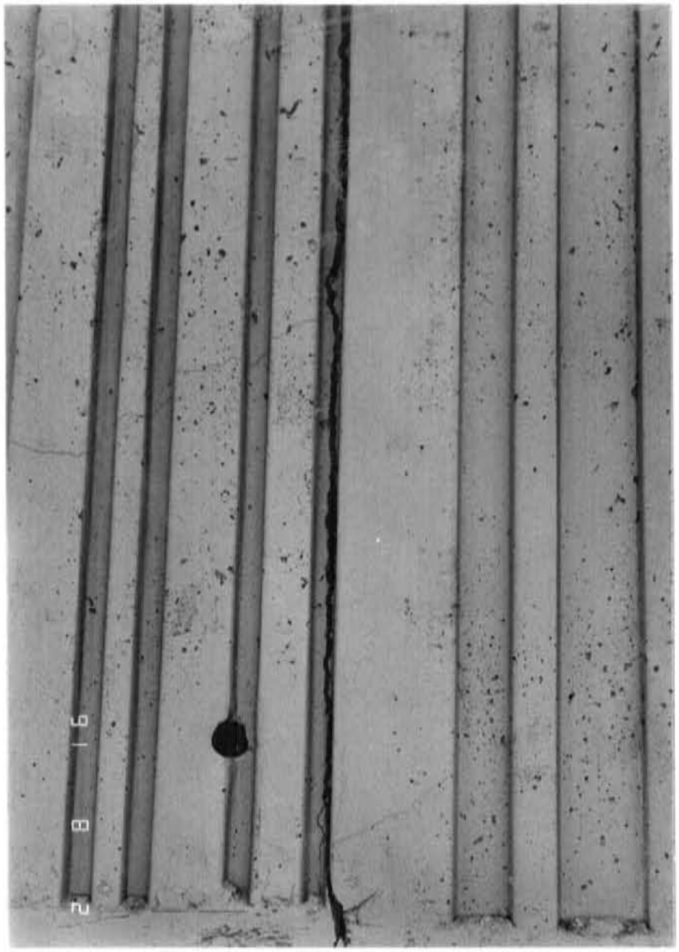


Figure 52 Large Vertical Crack Along the West Face of the Hilfiker Wall at the Moreland Street Drop Structure.

SPRR Overpass

The overpass of I-10 over the Southern Pacific Railroad (SPRR) is situated just south of the Washington-Jefferson Streets exit in Phoenix. The overpass is also referred to as the Harrison Street Bridge. There are two retaining wall systems here: a north wall and a south wall. The systems are reinforced earth retaining walls designed by VSL, and constructed in 1985. Figure 53 is a photograph of the south VSL wall at the SPRR overpass.

This site has been precisely surveyed by ADOT's Photogammetry and Mapping (P&M) Services, between June, 1988 and May, 1989, and again in September, 1991, in an endeavor to monitor the movements of the wall. The surveyors report that the wall has moved out and then back. The magnitudes of the movements have ranged from 0.02' to 0.05'.

Inspection of the VSL walls revealed many horizontal hairline cracks in the walls. The cracks appeared throughout the height of the walls, but were most frequent in the lowest one-third of the walls.

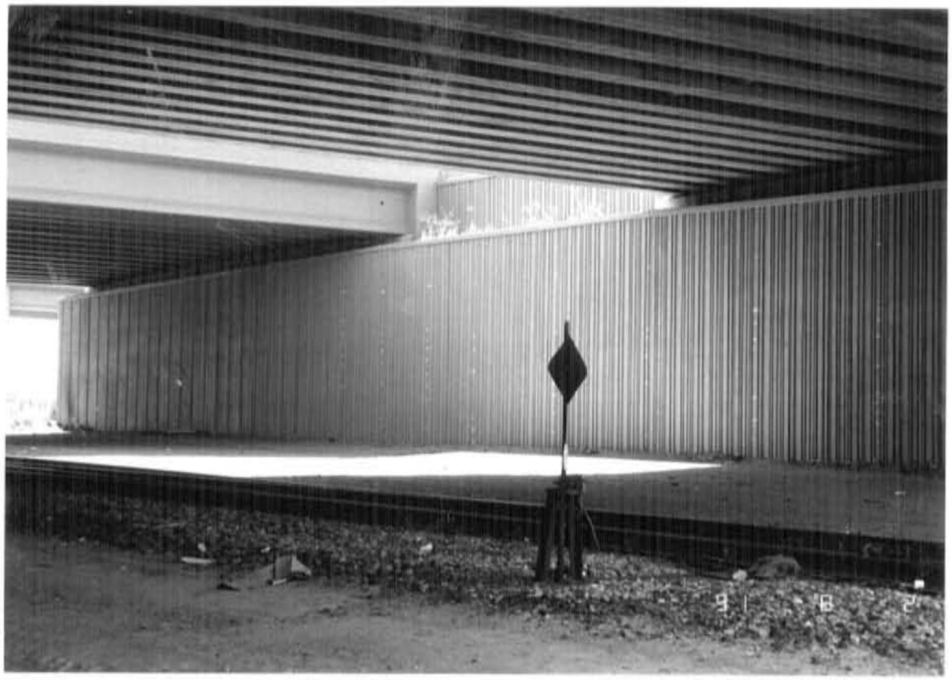


Figure 53 South Retaining Wall at the SPRR Overpass.

Warner Road Interchange

The retaining wall at the interchange of I-10 and Warner Road in Phoenix is also a Hilfiker wall and is located on the west side of the eastbound I-10 offramp. Figure 54 shows the geometry of the site. Because of the sound walls, the retaining wall is not visible from I-10 or Warner Road. The wall ranges in height linearly (north to south) 0' to 12.5'. The wall is not accessible in the northernmost 50' due to a chain link fence.

Inspection of this site consisted of walking the length of the retaining wall and recording the location of vertical and horizontal cracks, and expansion joints. The cracks were approximately 0.02" wide. The record is included as APPENDIX J. Figure 55 is a photograph of the retaining wall at the Warner Road Interchange.

There were plant wells placed at 8' intervals along the top of the retaining wall. Plants were no longer growing in these wells, but the drip irrigation system was continuing to operate. Also, irrigation of the plants immediately east of the sound wall was noticeably seeping down slope through the soil beneath the sound wall. The moisture was evident all the way down to the retaining wall. Because of the absence of weep holes in the retaining wall, the situation of additional water in the soil could be reducing the stability of the retaining wall. The drip irrigation system of the vacant plant wells has since been shut down. There were no major distresses visible on the wall.

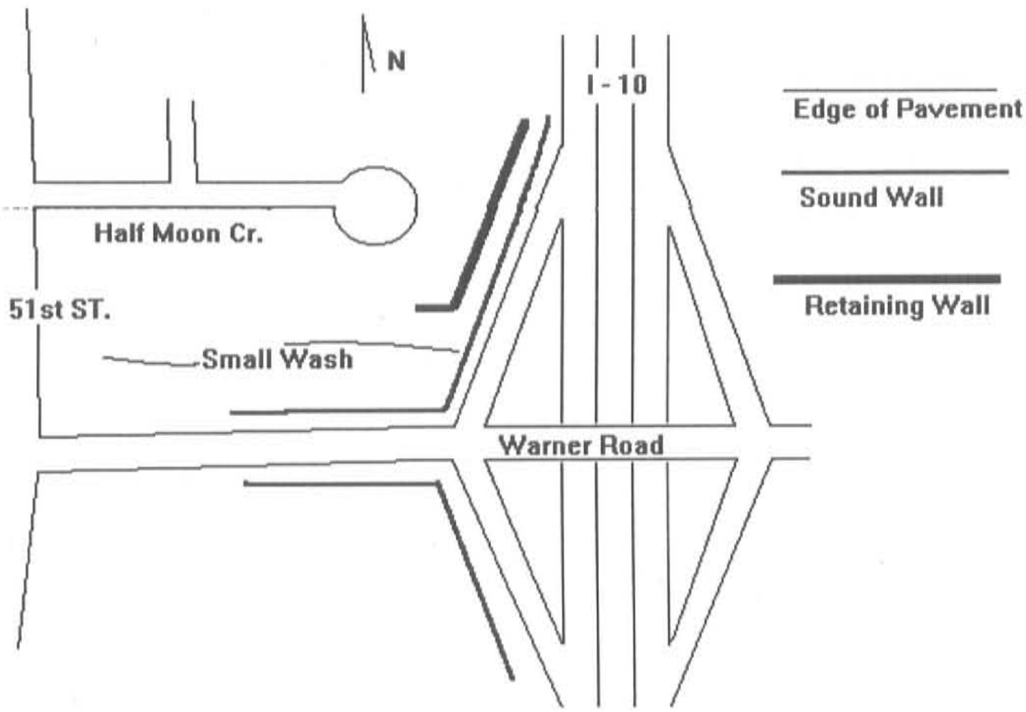


Figure 54 Location of the Warner Road I.C. Retaining Wall.

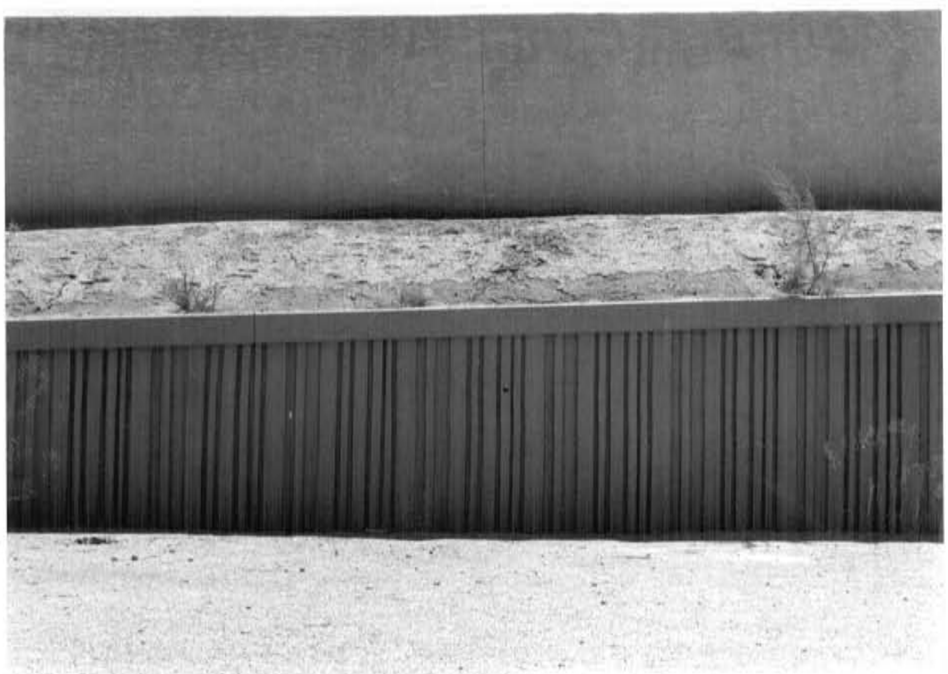


Figure 55 Hilfiker Wall at the Warner Road Interchange.

CONCLUSIONS

Currently the southeast Hilfiker wall of the experimental project (I-10 and 24th St.) is visibly distressed. There is a large vertical crack at the retaining wall - bridge abutment interface, and the PCCP anchor slab above this crack is deflecting noticeably. ADOT engineers have met to discuss the situation and formulate a plan of action.

A probable cause of the observed deflection of anchor slab is the formation of a void in the embankment. The void could be caused by the washing out of embankment material through the large vertical crack at the wall - abutment interface. This would be consistent with the washout problems encountered with the southwest Hilfiker wall during construction as seen in Figures 30, 31, and 32.

A hypothesis as to the cause of the large vertical crack is differential settlement between the bridge abutment and the Hilfiker wall/embankment. The bridge abutment is on piers, whereas the Hilfiker system is not.

Each of the other walls of this project are experiencing similar distress, but to a much lesser degree. The southwest and the northeast walls each have small cracks at their interface with the bridge abutments, and slight deflections of the anchor slabs have been noticed. These distresses are more significant on the southwest wall than the northeast wall.

ADOT has performed a Falling Weight Deflectometer (FWD) test at the site in an attempt to determine if a void is present, and if so, to what extent. The results indicate major deflections, supporting the void hypothesis. Coring through the PCCP will follow to confirm the presence of a void.

RECOMMENDATIONS

For future construction of reinforced earth systems, the following recommendations are made:

1. Backfill material used for similar projects should pass a 3" sieve. This recommendation is based on the experiences of this project. The finer material facilitated construction and was more readily compacted.

2. Compaction requirements for the backfill material should be increased from 90%, as specified for this project, to 95% of the maximum dry density. This may alleviate differential settlement between the area beneath the Hilfiker mats (90% compaction) and the rest of the embankment (95% compaction). This recommendation is consistent with ADOT Specification 205-3.04.

3. Better finish of the concrete panels may be achieved with a thicker wall. Construction of the forms for a thicker wall would allow more room for the use of vibrators. Also, forms constructed of steel, or a similarly rigid material would allow for better results of exterior vibrators.

4. In the design of a particular reinforced soil system, the features of the roadway should be considered. There should be no conflict between the soil reinforcement and the road features such as catch basins, anchor lugs, light pole foundations, etc.

5. Proper drainage should be provided for the retaining system. Drains should include sand or synthetic filters to prevent the loss of material from the embankment. This recommendation is in light of the extreme difficulty in keeping water from entering the embankment.

6. Continued visual monitoring and comparison surveys on the walls is recommended. Documentation of the methods used to correct existing distresses is also desirable.

REFERENCES

1. The University of Arizona, "Arizona Climate, The First Hundred Years," 1985.

2. Chart compiled from data from "Arizona Climate, The First Hundred Years," University of Arizona, 1985.

3. Chart compiled from data from "Arizona Climate, The First Hundred Years," University of Arizona, 1985.